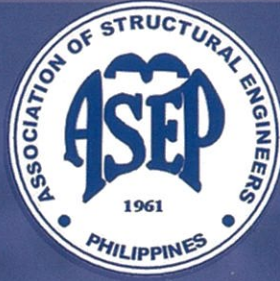


SEVENTH EDITION

ASSOCIATION OF STRUCTURAL ENGINEERS
OF THE PHILIPPINES, INC.



NATIONAL STRUCTURAL CODE OF THE PHILIPPINES 2015



VOLUME I
BUILDINGS, TOWERS, AND OTHER
VERTICAL STRUCTURES

C101-15

ISSN 2094-5477

7TH EDITION

NATIONAL STRUCTURAL CODE OF THE PHILIPPINES 2015

VOLUME I BUILDINGS, TOWERS AND OTHER VERTICAL STRUCTURES

SEVENTH EDITION
First Printing, 2016

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PREFACE TO THE NSCP VOLUME 1, SEVENTH EDITION, 2015

1. Introduction

ASEP recognizes the need for an up-to-date structural code addressing the design and installation of structural systems through requirements emphasizing performance. The new National Structural Code of the Philippines (NSCP Volume I) is designed to meet these needs through various model codes/regulations, generally from the United States, to safeguard the public health and safety nationwide.

This updated Structural Code establishes minimum requirements for structural systems using prescriptive and performance-based provisions. It is founded on broad-based principles that make possible the use of new materials and new building designs. Also, this code reflects the latest seismic design practice for earthquake-resistant structures.

2. Changes and Developments

In its drive to upgrade and update the NSCP, the ASEP Codes and Standards Committee initially wanted to adopt the latest editions of American code counterparts. However, for cases where available local data is limited to support the upgrade, then some provisions and procedures of the NSCP 7th edition were retained.

This NSCP 7th edition is referenced from the following:

- a. Uniform Building Code UBC-1997
- b. International Building Code IBC-2009
- c. American Society of Civil Engineers ASCE/SEI 7-10
- d. American Concrete Institute ACI318-14M
- e. American Institute for Steel Construction AISC-05 with Supplementary Seismic Provisions
- f. American Iron and Steel Institute AISI S100-2007
- g. Reinforced Masonry Engineering Handbook of America
- h. Concrete Masonry Handbook, 6th Edition
- i. American National Standard Institute ANSI EIA/TIA-222-G-I-2007
- j. American Society for Testing and Materials (ASTM) Standards

Significant revisions are summarized as follows:

- a. Chapter 1 – General Requirements

The changes made in this chapter are the following:

- a.1 Section 102 – Definition of Failure

- a.2 Section 103 – Classification of Structures

School buildings of more than one story, hospitals, designated evacuation centers, structures are under the essential facilities category. Section 104 – Design Requirements

Churches, Mosque and other related religious structures are under the special occupancy category Section 104-Design Requirements.

The provision for deflection of any structural member under the serviceability requirement is deleted. This requirement for concrete and steel is specified in Chapters 4 and 5 respectively.

New requirements are added to the design review section.

- a.3 Section 105 – Posting and Instrumentation

The provision of installed recording accelerograph is adjusted.

- a.4 Inclusion of Appendix 1-A : Recommended Guidelines on Structural Design Peer Review of Structures 2015
- a.5 Inclusion of Appendix 1-B: Guidelines and Implementing Rules on Earthquake Recording Instrumentation for Buildings

b. Chapter 2 – Minimum Design Loads

The changes made in this chapter are the following:

b.1 Section 203 – Combination of Loads

The load factors and load combinations are revised particularly the load combinations including wind load.

b.2 Section 205 – Live Loads

Additional loads are incorporated in the table for minimum uniform and concentrated loads particularly the parking garage and ramp live load.

b.3 Section 207 – Wind Loads

Wind load provisions, which were previously based on ASCE7-05, are updated based on ASCE7-10. In this edition, three different wind contour maps for the entire Philippine archipelago are generated and provided for determining the basic wind speeds for different categories of building occupancies as defined in Table 103-1. These maps provide basic wind speeds that are directly applicable for determining pressures for design strength. Strength design wind load factor is 1.0; whereas, allowable stress design wind load factor is 0.6. Generally, basic wind speeds correspond to 3%, 7% and 15% probability of exceedance in 50 years ($MRI = 1700, 700$ and 300 years, respectively). Four (4) permitted procedures in determining the design wind loads for main wind-force resisting systems (MWFRS), for other structures and building appurtenances and for components and cladding (C&C) are provided such as;

- directional procedure for buildings of all heights,
- envelope procedure for low-rise buildings,
- directional procedure for other structures and building appurtenances and analytical procedure for components & cladding, and
- wind tunnel procedure

The ANSI EIA/TIA-222-G-2005 and ANSI EIA/TIA-222G-1-2007 are now fully referenced for computing wind loads on steel antenna towers and antenna supporting structures.

b.4 Section 208 – Earthquake Loads

The near-source factors for 2-km distance from a causative fault is included in addition to 5-km, 10-km, 15-km distance and beyond 15-km distance.

ASCE/SEI 7-10, using spectral acceleration, is recognized as an alternative procedure in the determination of the earthquake loads.

c. Chapter 3- Earthworks and Foundations

The revisions made in this chapter are the following:

- c.1 Provisions pertaining to the conduct and interpretation of foundation investigations for cases involving liquefiable, expansive or questionable soils are adopted;

- c.2 The section on footings is amended to incorporate provisions for differential settlement, design loads and vibratory loads;
- c.3 The section on pile foundations is amended to incorporate new provisions on splicing of concrete piles; and
- c.4 The section on special foundations, slope stabilization and materials of construction are added.
- c.5 Provisions for construction in Zone 4 pertaining to reinforcement of Precast Prestressed Piles have been revised to ensure consistency with ACI 318.
- c.6 The figure for cut slopes has been amended for clarity;
- c.7 The figure for fill slopes has been amended for clarity and some provisions have been modified;
- c.8 A table on the minimum required number of boreholes has been added to the section on foundation investigation;
- c.9 Provisions pertaining to minimum dimensions of ditches have been modified;
- c.10 The section on excavations and fills has been amended to incorporate provisions for scouring and erosion protection as well as support of excavations and open cuts;
- c.11 Provision pertaining to general pile requirements have been expanded to include design of piles and pile groups subjected to lateral loads.
- c.12 A Section on MSE Structures and Similar Reinforced Embankments and Fills has been added.

d. Chapter 4 - Structural Concrete

To reflect the reorganization of ACI 318-14 which contained a number of significant technical changes, the ASEP adopted similar changes in the NSCP 2015 7th Edition. The latest ACI 318 was reorganized as a member-based document, i. e., particular member type, such as beam, column, or slab will have separate sub-sections for all requirements to design that particular member type. This will eliminate the need to flip through several Sections to comply with all the necessary design requirements for a particular structural member, as was necessary with the old organization format.

d.1 Section 401: General

General information regarding the scope and applicability of NSCP 2015, Vol. 1 is provided. Additional sub-section on interpretation is included to help users better understand Chapter 4, Structural Concrete.

d.2 Section 402: Notation and Terminology

The definition for hoops has been modified because the use of interlocking headed bars is a concern regarding the possibility that it will not be adequately interlocked and because the heads could become disengaged under complex loadings well into the non-linear range of response. It is now defined as a closed tie or continuously wound tie, made up of one or several reinforcement elements, each having seismic hooks at both ends.

A definition for special seismic systems, a term used in Sections 418 and 419, has been added.

d.3 Section 403: Referenced Standards

The following referenced specifications have been added to Section 403.2.4:

- ASTM A370-14, Standard Test Methods and Definitions for Mechanical Testing of Steel Products
- ASTM A1085-13, Standard Specification for Cold-Formed Welded Carbon Steel Hollow Structural Sections (HSS)
- ASTM C173/C173M-14, Standard Test Method for Air- Content of Freshly Mixed Concrete by Volumetric Method

- ASTM C1582/C1582M-11, Standard Specification for Admixtures to Inhibit Chloride-Induced Corrosion of Reinforcing Steel in Concrete

A new referenced specification from Australia and New Zealand, Section 403.2.6 is added. These standards were included as ACI 318 has no provisions related to Qualifications on the Use of Quenched Tempered QT/Thermo-Mechanically Treated Reinforcement, which are the type manufactured, sold, and commonly used for building construction in the Philippines:

- AS/NZS 4671: 2001, Steel Reinforcing Materials
- NZS 3101: 2006, Part 1 and Part 2, Concrete Structures Standard, and Design of Concrete Structures
- NZS 3109, Amendment 2, Welding of Reinforcing Steel
- AS/NZS 1554.3: 2008, Part 3, Structural Steel Welding of Reinforcing Steel

The following referenced specifications have been deleted:

- ASTM C109/C109M-08, Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 50 mm Cube Specimens)
- ASTM C192/C192M-07, Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory

Several referenced standards and specifications have been updated, as in most cases with every edition of the NSCP. Note that the edition of every referenced standard is important. The NSCP does not necessarily adopt new editions of referenced standards unless they are vetted before the publication of each edition of the standard.

d.4 Section 404: Structural System Requirements

This new Section has been added to Chapter 4 to introduce structural system requirements. This Section contains Sub-sections on Materials, Design Loads, Structural System and Load Paths, Structural Analysis, Strength, Serviceability, Durability, Sustainability, Structural Integrity, Fire Resistance, Requirements for Specific Types of Construction, Construction and Inspection, and Strength Evaluation of Existing Structures. Most of these Sub-sections refer to the other Sections in the NSCP. The Sub-section on construction and inspection, for instance, refers to Section 426. In the areas for Sustainability and Fire Resistance, the NSCP does not have specific requirements. This Sub-section on Sustainability allows the licensed design professional to specify in the construction documents, sustainability requirements in addition to the strength, serviceability, and durability requirements of the NSCP. The strength, serviceability, and durability requirements are required to take precedence over sustainability considerations, though these requirements are generally in harmony with sustainable structures. In the Sub-section on Fire Resistance, the NSCP refers to the fire-protection requirements of the NSCP Chapter 4, Sub-section 420.6.1. However, if the National Building Code of the Philippines requires a greater concrete cover, such greater thickness shall govern.

d.5 Section 405: Loads

The following modification has been made in the provision for live load reduction because there are still unincorporated areas where there may not be included in the previous editions of the NSCP. The 7th Edition, Sub-section 405.2.3 – Live load reductions shall be permitted in accordance with the National Building Code of the Philippines, or in its absence, in accordance with ASCE/SEI 7.

For many Code revision cycles, ACI 318 retained provisions for service-level earthquake forces in the design load combinations. In 1993, ASCE/SEI 7 converted earthquake forces to strength-level forces and reduced the earthquake load factor to 1.0, and the model building

codes followed suit. In modern building codes around the world, earthquake loads are now strength-level forces. Any references to service-level earthquake forces have been deleted.

d.6 Section 406: Structural Analysis

The following new item has been added in Sub-section 406.6.2.3:

(b) For frames or continuous construction, it shall be permitted to assume the intersecting member regions are rigid.

Previous NSCP 6th Edition has been silent on the use of finite element analysis (FEA), though it is now frequently used. Sub-section 406.9 now has provisions that are intended to explicitly allow the use of FEA and to provide a framework for the future expansion of FEA provisions, but not as a guide toward the selection and use of FEA software. The new Sub-section on diaphragms and collectors makes explicit reference to the use of FEA, which makes it imperative that the NSCP 7th Edition recognize the acceptability of its use.

d.7 Section 408: Two-Way Slabs

Sub-section 418.10.1 (corresponding to ACI 318M-11, Section 18.9.1), says that a minimum area of bonded reinforcement shall be provided in all flexural members with unbonded prestressing tendons. The purpose of the minimum unbonded reinforcement over the tops of columns is to distribute cracking caused by high local flexural tensile stresses in areas of peak negative moments. However, the high local flexural tensile stresses are not unique to slabs with unbonded tendons. The new reorganized Sub-section 408.6.2.3 (corresponding to ACI 318M-14 Section 8.6.2.3) requires the same minimum reinforcement in slabs with unbonded or bonded tendons, except that the area of bonded tendons is considered effective in controlling cracking.

It was also decided by the ACI 318 Committee, that if the same bonded reinforcement were required for both bonded and unbonded post-tensioned two-way systems, the structural integrity requirements for both systems should also be the same. The structural integrity requirements in ACI 318M-11, Section 18.12.6 applied to two-way post-tensioned slab systems with unbonded tendons only. The structural integrity requirements in ACI 318M-14 Section 8.7.5.6 (corresponding to the NSCP 2015, Sub-section 408.7.5.6) now apply to two-way post-tensioned slab systems with bonded as well as unbonded tendons.

d.8 Section 409: Beams

The use of open web reinforcement for torsion and shear in slender spandrel beams by the precast concrete industry as an alternative to the closed stirrups traditionally mandated by this Code. Eliminating closed stirrups is desirable because they cause reinforcement congestion; production costs also increase significantly because pre-tensioning strand must be threaded through the closed stirrups.

A new relevant Sub-section 409.5.4.7 for solid precast sections is added to the NSCP 7th Edition.

d.9 Section 412: Diaphragms

For the first time, a new Section 412, added design provisions for diaphragms in buildings constructed in areas of low seismicity (Zone 2) The new Section applies “to the design of non-prestressed and prestressed diaphragms, including:

- (a). Diaphragms that are cast-in-place slabs
- (b). Diaphragms that comprise a cast-in-place topping slab on precast elements

(c). Diaphragms that comprise precast elements with end strips formed by either a cast-in-place concrete topping slab or edge beams

(d). Diaphragms of interconnected precast elements without cast-in-place concrete topping

d.10 Section 418: Earthquake-Resistant Structures

There are a number of significant and substantive changes to this Section.

Column confinement - The ability of the concrete core of a concrete reinforced column to sustain compressive strains tends to increase with confinement pressure. Confinement requirements for columns of special moment frames, and for columns not designated as part of the seismic-force-resisting system in structures assigned to seismic zone 4 (similar to ASCE 7-10 Seismic Design Categories D, E, and F), with high axial load or high concrete compressive strength are significantly different.

Transverse reinforcement - One important new requirement for special moment frame columns are included in Sub-sections 418.7.5.2 and 418.7.5.4. There are new restrictions on the use of headed reinforcement to make up hoops.

Special moment frame beam-column joints - For beam-column joints of special moment frames, clarification of the development length of the beam longitudinal reinforcement that is hooked, requirements for joints with headed longitudinal reinforcement, and restrictions on joint aspect ratio are new. For beam-column joints of special moment frames, clarification of development length of beam longitudinal reinforcement that is hooked, requirements for joints with headed longitudinal reinforcement, and restrictions on joint aspect ratio are new.

Special shear walls - Subsection 418.10 (equivalent to ACI 318-14M-14 Section 18.10, previously ACI 318M-11 Section 21.9), has been extensively revised in view of the performance of buildings in the Chile earthquake of 2010 and the Christchurch, New Zealand, earthquakes of 2011, as well as full-scale reinforced concrete building tests. In these earthquakes and laboratory tests, concrete spalling and vertical reinforcement buckling were at times observed at wall boundaries.

For ASTM A615 Grade 420 bars used as longitudinal reinforcement in special moment frames and special shear walls, the NSCP 7th Edition now requires the same minimum elongation as ASTM A706 reinforcement.

d.11 Section 419: Concrete: Design and Durability Requirements

Quite a few changes have been made in concrete durability requirements, which are now located in this Section.

d.12 Section 420: Steel Reinforcement Properties, Durability and Embedments

The definition of yield strength of high-strength reinforcement for Grade 420 (Grade 60) in this Section is now, for the first time, the same as that in ASTM specifications, except for bars with less than 420 MPa, the yield strength shall be taken as the stress corresponding to a strain of 0.35 percent.

Deformed and plain stainless steel wire and welded wire conforming to ASTM A1022 is now permitted to be used as concrete reinforcement.

Sub-section 420.2.2.5 requires "Deformed non-prestressed longitudinal reinforcement resisting earthquake moment, axial force, or both, in special moment frames, special structural walls, and all the components of special structural walls including coupling beams and wall piers" to be ASTM A706 Grade 420 (Grade 60), ASTM 615 Grade 275 (Grade 40) or Grade 420 (Grade 60) reinforcement is permitted if two supplementary requirements are met, which are already part of the ASTM A706 specification. A third supplementary requirement is now added for ASTM A615 (Grade 60) reinforcement to be permitted for use in special moment frames, special structural walls. The minimum elongation in 200 mm (8") must now be the same as that ASTM A615 (Grade 60) reinforcement.

One aspect of the Code compliance that the Association of Structural Engineers of the Philippines is cautioning Designers and Constructors alike, is the introduction of ASTM 615 Grade 520 (Grade 75) in the Philippine

market. Since this was not covered by previous editions of the NSCP Vol. 1, it creates an impression of an unregulated use of a new high-strength reinforcement grade. NSCP 7th Edition.

To put it clearly, Sub-section 420.2.2.5, corresponding to ACI 318M-14 Section 20.2.2.5, specifies the use of deformed non-prestressed longitudinal reinforcement resisting earthquake-induced moment, axial force, or both, in special moment frames, special structural walls, and all components of special structural walls, including coupling beams, and wall piers which shall be in accordance with (a) or (b):

(a). ASTM A706M, Grade 420

(b). ASTM A615M, Grade 280

There was no mention that ASTM A615M, Grade 520, was allowed, although the use of micro-alloyed high-strength reinforcement may be allowed in the future through the issuance of a new ASTM or updated standard, and with proper validation by the Department of Trade and Industry's Bureau of Standards. It will be premature to allow its use for special moment frames, special structural walls, and all components of special structural walls, including coupling beams, and wall piers for Buildings located in areas of high seismicity (zone 4). The same restrictions indicated in Sub-section 420.7.6, on the use of quenched-tempered thermo-mechanically treated (QT/TMT) reinforcing bars in structures located in seismic zone 4 for Grade 420 reinforcement, shall also be applied to Grade 520, unless proven in subsequent studies and tests.

d.13 Section 422: Sectional Strength

The following are the changes in Section 422:

For prestressed members, a new equation for the nominal axial strength at zero eccentricity has been introduced in Sub-section 422.4.2.3.

New Sub-section 422.4.3.1, which requires that the nominal axial tensile strength of a non-prestressed, composite, or prestressed member, not to be taken greater than the maximum nominal axial tensile strength of member.

d.14 Section 425: Reinforcement Details

Two changes shown in Table 7 (part of Table 425. 3.2) are made to eliminate the differences between the required tail extension of a 90-degree or 135- degree standard hook, subject to a minimum of 75 mm (3").

Mechanical or welded splices with strengths below 125% of the yield strength of the spliced reinforcing bars are no longer permitted. The associated stagger requirements have been deleted. Thus there is no longer a need to specify "full" mechanical or "full" welded splices.

d.15 Section 426: Construction Documents and Inspection

In this section, the user will probably require some time to get used to, it starts with the following:

426.1.1 This Sub-section addresses (a) through (c):

- (a) Design information that the licensed design professional shall specify in the construction documents,
- (b) Compliance requirements that the licensed design professional shall specify in the construction documents,
- (c) Inspection requirements that the licensed design professional shall specify in the construction documents,

Thus, construction and inspection requirements have been consolidated, and they are now related to construction documents. The construction requirements are designated either as "design information" or "compliance requirements." These are largely existing material that has been rearranged. The

inspection requirements in Sub-section 426.13 are taken from Chapter 17 of the 2015 *International Building Code* (IBC) and were previously not part of ACI 318.

Provisions in ACI 318-11 and earlier editions, which explained basic statistical considerations in mixture proportioning, are no longer found in ACI 318-14. Instead, ACI 301-10, *Specifications for Structural Concrete*, is referenced.

These are some other changes in the makeup of NSCP 2016 7th Edition that should be noted:

1. There are two new Sections: Section 404, Structural System Requirements and Section 412, Diaphragms.
2. Section 422, Structural Plain Concrete, now Section 414.
3. Section 423, Anchoring to Concrete, is now Section 417, with no significant changes.
4. Section 421, Earthquake-Resistant Structures, now Section 418.
5. Section 427, Strut-and-Tie Models is now Section 423, with no significant changes.
6. Section 420, Strength Evaluation of Existing Structures, is now Section 427.
7. Section 419, Shells and Folded Plates, is now Section 428.
8. Section 424, Alternative Design Method, now Section 429, is adapted from earlier editions of the NSCP.
9. Section 425, Alternative Provisions for Reinforced and Prestressed Concrete Flexural and Compression Members, and Section 426, Alternative Load and Strength Reduction Factors, have been discontinued.
10. On the other hand, Section 416, Precast Concrete, and Section 418, Prestressed Concrete, no longer exist as separate entities. The provisions of these Sections are now spread over several of the new Sections.

Sub-section 418.18, Requirements for post-tensioning ducts and grouting have also been removed as being outdated. The Commentary now provides specification guidance.

e. Chapter 5: Structural Steel.

ASEP adapted the American Institute of Steel Construction (AISC) 14th Edition in this updated Structural Steel code. The revisions made in this chapter are the following:

e.1 The entire Structural Steel chapters are streamlined placing all chapter definitions under one Definition heading, tables are immediately shown where they are first mentioned, figures drawn larger, equation are all in boldface, extraneous user notes are removed, essential in-text definitions italicized and in-text equation terms are written in boldface for easy reference.

e.2 Change of headings and terms.

501.3.5 Filler Metal and Flux for Welding to 501.3.5 Consumables for Welding

510.10.3 Web Crippling to Web Local Crippling

557.5 Special Fabrication Requirements. Weld tabs changed to Run-off tabs under Exception.

A-6.3 Beams changed to Beams Bracing

e.3 Creation of new subtopic.

APPENDIX A-4

STRUCTURAL FIRE

A-4.2.3.1 Thermal Elongation is created under A-4.2.3 Material Strengths at Elevated Temperatures

APPENDIX A-6

STABILITY BRACING FOR COLUMNS AND BEAMS

A-6.4 Beam-Column Bracing

SECTION 529 BUCKLING-RESTRAINED BRACED FRAMES (BRBF)

A section 529.3 was created as heading for 529.3.1 and 529.3.2

e.4 Revision in load factor

510.8 Column Bases and Bearing on Concrete

$$2010: \phi_c = 0.60(LRFD) \quad \Omega_c = 2.5(ASD)$$

$$2015: \phi_c = 0.65(LRFD) \quad \Omega_c = 2.31(ASD)$$

e.5 Revision in equations

B-5. QUALIFYING CYCLIC TESTS OF BUCKLING-RESTRAINED BRACES

511.2.2c Branches with Axial Loads in K-Connections

$$2010: Q_r = \gamma^2 \left[1 + \frac{0.24\gamma^{1.2}}{e^{\left(\frac{0.5g}{r} - 1.33\right)} + 1} \right] \quad (511.2-7) \quad ; 2015: Q_r = \gamma^{0.2} \left(1 + \frac{0.24\gamma^{1.2}}{e^{\left(\frac{0.5g}{r} - 1.33\right)} + 1} \right) \quad (511.2-7)$$

APPENDIX A-3 - DESIGN FOR FATIGUE

A-3.4 Bolts and Threaded Parts

$$2010: A_t = \frac{\pi}{4} (d_b - 9382)^2 \quad ; 2015: A_t = \frac{\pi}{4} (d_b - 0.9382P)^2$$

f. Chapter 6: Wood.

The revisions made in this Chapter are the following:

- f.1 Section 616 – Design Provisions and Equations: The NDS 2015 Chapter 3 is adopted almost in its entirety;
- f.2 Section 617 – Sawn Lumber: The NDS 2015 Chapter 4 is adopted almost in its entirety;
- f.3 Section 618 – Structural Glued Laminated Timber: The NDS 2015 Chapter 5 is adopted almost in its entirety;
- f.4 Tables 619.1-3 and 619.1-4 are revised based on NDS 2015; and
- f.5 Other Sections affected are adjusted accordingly.

g. Chapter 7: Masonry

The revisions made in this chapter are the following:

- g.1 The specified yield strength of steel reinforcement is 420MPa instead of 413 MPa / 415 MPa;

- g.2 Section 710.6.3 and Section 710.7.1
 10mm diameter instead of No.9 gage wire
- g.3 Section 713.9.1
 20mm diameter instead of 19-gage
 25mm diameter instead of 24-gage

3. **Acknowledgment**

The ASEP Codes and Standards Committee are indebted to Philippine Institute of Volcanology and Seismology (PHIVOLCS) and to Dir. Renato V. Solidum, Ph. D. for his unselfish contribution specifically on Section 208 of this code.

ASEP acknowledges the contribution of Dr. Teresito C. Bacolcol and Ms. Madeline Cabologan of PHIVOLCS for the seismic maps used in this code.

ASEP acknowledges the contribution of Engr. Carlos M. Villaraza for his unselfish contribution on Chapter 2 Seismic/Earthquake Chapter.

The contributions of ASEP members and other users of this code who have suggested improvements, identified errors and recommended items are recognized.

ASEP also acknowledges the contribution of the industry partners, companies and individuals, who continue to support ASEP's numerous undertakings.

The ASEP Codes and Standards Committee also acknowledge Arch. Avigaile Genota Riola who designs the covers of the NSCP Volume 1, 2010 Edition and NSCP Volume 1, 2015 Edition.

Chapter 1

GENERAL REQUIREMENTS

NATIONAL STRUCTURAL CODE OF THE PHILIPPINES VOLUME I BUILDINGS, TOWERS AND OTHER VERTICAL STRUCTURES

SEVENTH EDITION, 2015

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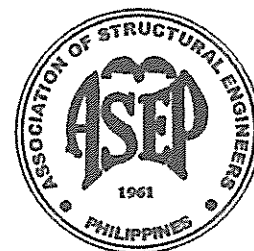


Table of Contents

SECTION 101	2
TITLE, PURPOSE AND SCOPE.....	2
101.1 Special Foundation Systems.....	2
101.2 Purpose.....	2
101.3 Scope.....	2
101.4 Alternative Systems.....	2
SECTION 102	3
DEFINITIONS.....	3
SECTION 103	6
CLASSIFICATION.....	6
OF STRUCTURES.....	6
103.1 Nature of Occupancy.....	6
SECTION 104	7
DESIGN REQUIREMENTS	7
104.1 Strength Requirement.....	7
104.2 Serviceability Requirement	7
104.3 Analysis.....	7
104.4 Foundation Investigation.....	8
104.5 Design Review	8
SECTION 105	9
POSTING AND INSTRUMENTATION.....	9
105.1 Posting of Live Loads.....	9
105.2 Earthquake-Recording Instrumentation.....	9
SECTION 106	9
SPECIFICATIONS, DRAWINGS AND CALCULATIONS	9
106.1 General.....	9
106.2 Specifications	9
106.3 Design Drawings.....	9
SECTION 107	10
STRUCTURAL INSPECTIONS, TESTS AND STRUCTURAL OBSERVATIONS.....	10
107.1 General.....	10
107.2 Definitions.....	10
107.3 Structural Inspector	10
107.4 Inspection Program	11
107.5 Types of Work for Inspection	11
107.6 Approved Fabricators.....	13
107.7 Prefabricated Construction	13
107.8 Non-Destructive Testing	14
107.9 Structural Observation.....	15
APPENDICES	
I-A – Recommended Guidelines on Structural Design Peer Review of Structures 2015	IA-1
I-A – Guidelines and Implementing Rules on Earthquake Recording Instrumentation for Buildings.....	IB-1

SECTION 101

TITLE, PURPOSE AND SCOPE

101.1 Special Foundation Systems

These regulations shall be known as the **National Structural Code of the Philippines 2015, Volume I, 7th Edition**, and may be cited as such and will be referred to herein as "this code."

101.2 Purpose

The purpose of this code is to provide minimum requirements for the design of buildings, towers and other vertical structures, and minimum standards and guidelines to safeguard life or limb, property and public welfare by regulating and controlling the design, construction, quality of materials pertaining to the structural aspects of all buildings and structures within this jurisdiction.

101.3 Scope

The provisions of this code shall apply to the construction, alteration, moving, demolition, repair, maintenance and use of buildings, towers and other vertical structures within this jurisdiction.

Special structures such as but not limited to single family dwellings, storage silos, liquid product tanks and hydraulic flood control structures, should be referred to special state of practice literature but shall refer to provisions of this code as a minimum wherever applicable.

For additions, alterations, maintenance, and change in use of buildings and structures, see Section 108.

Where, in any specific case, different sections of this code specify different materials, methods of construction or other requirements, the most restrictive provisions shall govern except in the case of single family dwellings. Where there is a conflict between a general requirement and a specific requirement, the specific requirement shall be applicable.

101.4 Alternative Systems

The provisions of this code are not intended to prevent the use of any material, alternate design or method of construction not specifically prescribed by this code, provided any alternate has been permitted and its use authorized by the Building Official (see Section 102).

Sponsors of any system of design or construction not within the scope of this code, the adequacy of which had been shown by successful use and by analysis and test, shall have the right to present the data on which their design is based to the Building Official or to a board of examiners appointed by the Building Official or the project owner/developer. This board shall be composed of competent structural engineers and shall have authority to investigate the data so submitted, to require tests if any, and to formulate rules governing design and construction of such systems to meet the intent of this code. These rules, when approved and promulgated by the Building Official, shall be of the same force and effect as the provisions of this code

SECTION 102 DEFINITIONS

For the purpose of this code, certain terms, phrases, words and their derivatives shall be construed as specified in this chapter and elsewhere in this code where specific definitions are provided. Terms, phrases and words used in the singular include the plural and vice versa. Terms, phrases and words used in the masculine gender include the feminine and vice versa.

The following terms are defined for use in this code:

ALTER or ALTERATION is any change, addition or modification in construction or occupancy.

APPROVAL shall mean that the proposed work or completed work conforms to this code in the opinion of the Building Official.

APPROVED as to materials and types of construction, refers to approval by the Building Official as the result of investigation and tests conducted by the Building Official, or by reason of accepted principles or tests by recognized authorities, technical or scientific organizations.

AS GRADED is the extent of surface conditions on completion of grading.

AUTHORITY HAVING JURISDICTION is the organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this code.

BEDROCK is in-place solid or altered rock.

BENCH is a relatively level step excavated into earth material on which fill is to be placed.

BORROW is earth material acquired from an off-site location for use in grading on a site.

BUILDING is any structure usually enclosed by walls and a roof, constructed to provide support or shelter for an intended use or occupancy.

BUILDING, EXISTING is a building erected prior to the adoption of this code, or one for which a legal building permit has been issued.

BUILDING OFFICIAL is the officer or other designated authority charged with the administration and enforcement of this code, or the Building Official's duly authorized representative.

CIVIL ENGINEER is a professional engineer licensed to practice in the field of civil engineering.

CIVIL ENGINEERING is the science or profession in which a knowledge of the mathematical and physical sciences gained by study and practice is applied with judgement to utilize natural and man-made resources and forces in the planning, design, management, construction, and maintenance of buildings, structures, facilities, and utilities in their totality, for the progressive well-being and for the benefit of mankind, enhancing the environment, community living, industry, and transportation, taking into consideration such aspects as functionality, efficiency, economy, safety, and environmental quality.

COMPACTION is the densification of a fill by mechanical or chemical means.

CONSTRUCTION FAILURE is a failure that occurs during construction and they are considered to be either a collapse or distress, of a structural system to such a degree that it cannot safely serve its intended purpose.¹

CONTINUOUS STRUCTURAL INSPECTION is a structural inspection where the structural inspector is on the site at all times observing the work requiring structural inspection.

EARTH MATERIAL is any rock, natural soil or fill or any combination thereof.

ENGINEER-OF-RECORD is a civil engineer responsible for the structural design.

EROSION is the wearing away of the ground surface as a result of the movement of wind or water.

EXCAVATION is the mechanical removal of earth material.

EXISTING GRADE is the grade prior to grading.

FAILURE is defined as an unacceptable difference between expected and observed performance. This definition includes catastrophic structural collapse, but also includes performance problems that are not necessarily catastrophic or life-threatening, including "serviceability problems such as distress, excessive deformation, premature deterioration of materials, leaking roofs and facades, and inadequate interior environmental control systems." In the event of a significant failure, the parties typically retain experts to determine the cause of the perceived failure. Occasionally a failure results from a

single condition, but typically, failures result from a combination of mistakes, oversights, miscommunications, misunderstandings, ignorance, lapses, slips, incompetence, intentional violations or non-compliance, and inadequate quality assurance. The causes for these conditions vary, but may include simple mistakes (such as sending information to a structural engineer when it should have been sent to the architect), conclusions based on faulty assumptions, an employee's "laziness, ignorance, or malevolent urge," fatigue from excessive workload, inadequate training, "time boxing" practices used to minimize fees to a client, overreliance on computer-aided design and drafting (CADD), failure to understand and deliver client requirements, time pressures to a deliver a project by certain deadlines, and ineffective coordination and integration of the design team.²

FILL is a deposit of earth material placed by artificial means.

FINISH GRADE is the final grade of the site that conforms to the approved plan.

FORENSIC ENGINEERING is the application of the art and science of engineering in the jurisprudence system, requiring the services of legally qualified engineers. Forensic engineering may include investigation of the physical causes of accidents and other sources of claims and litigation, preparation of engineering reports, testimony at hearings and trials in administrative or judicial proceedings, and the rendition of advisory opinions to assist the resolution of disputes affecting life or property.³

GENERAL COLLAPSE is the immediate, deliberate demolition of an entire structure by a triggering event (e.g. explosion).⁴

GEOTECHNICAL ENGINEER is a registered Civil Engineer with special qualification in the practice of Geotechnical Engineering as recognized by the Board of Civil Engineering of the Professional Regulation Commission as endorsed by the Specialty Division of Geotechnical Engineering of the Philippine Institute of Civil Engineers (PICE).

GEOTECHNICAL ENGINEERING is the application of the principles of soil and rock mechanics in the investigation, evaluation and design of civil works involving the use of earth materials and foundations and the inspection or testing of the construction thereof.

GRADE is the vertical location of the ground surface.

GRADING is an excavation or fill or combination thereof.

KEY is a designed compacted fill placed in a trench excavated in earth material beneath the toe of a slope.

LIMITED LOCAL COLLAPSE is a failure of a structural member without affecting the adjacent members (e.g. destruction of one or two columns in a multi-bay structure).⁴

OCCUPANCY is the purpose for which a building or other structures or part thereof, is used or intended to be used.

PERIODIC STRUCTURAL INSPECTION is a structural inspection where the inspections are made on a periodic basis and satisfy the requirements of continuous inspection, provided this periodic scheduled inspection is performed as outlined in the inspection program prepared by the structural engineer.

PREFABRICATED ASSEMBLY is a structural unit, the integral parts of which have been built up or assembled prior to incorporation in the building.

PROFESSIONAL INSPECTION is the inspection required by this code to be performed by the civil engineer. Such inspections include that performed by persons supervised by such engineer and shall be sufficient to form an opinion relating to the conduct of the work.

PROGRESSIVE COLLAPSE is the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or disproportionately large part of it.⁴

ROBUSTNESS is the insensibility of a structure to local failure. From this definition, it follows that the robustness is a property of the structure.⁴

ROUGH GRADE is the stage at which the grade approximately conforms to the approved plan.

SITE is any lot or parcel of land or contiguous combination thereof, under the same ownership, where grading is performed or permitted.

SLOPE is an inclined ground surface the inclination of which is expressed as a ratio of vertical distance to horizontal distance.

SOIL is naturally occurring superficial deposits overlying bedrock.

SOILS ENGINEER See Geotechnical Engineer.

SOILS ENGINEERING See Geotechnical Engineering.

STRUCTURE is that which is built or constructed, an edifice or building of any kind, or any piece of work artificially built up or composed of parts joined together in some definite manner.

STRUCTURAL ENGINEER is a registered Civil Engineer with special qualification in the practice of Structural Engineering as recognized by the Board of Civil Engineering of the Professional Regulation Commission or by the Specialty Division of the Philippine Institute of Civil Engineers (PICE) together with the Association of Structural Engineers of the Philippines (ASEP) and Institution of Specialist Structural Engineers of the Philippines (ISSEP).

STRUCTURAL ENGINEERING is a discipline of civil engineering dealing with the analysis and design of structures that support or resist loads insuring the safety of the structures against natural forces.

STRUCTURAL FAILURE is the reduction of capability of a structural system or component to such a degree that it cannot safely serve its intended purpose.⁵

Structural failures can be divided into various categories based on consequential damages to include: Catastrophic Failure with Loss of Life, Catastrophic Failure in which No Human Lives are Endangered, Failure Resulting in Extensive Property Damage, and Failure Resulting in Reduced Serviceability.⁵

STRUCTURAL INSPECTION is the visual observation by a structural inspector of a particular type of construction work or operation for the purpose of ensuring its general compliance to the approved plans and specifications and the applicable workmanship provisions of this code as well as overall construction safety at various stages of construction.

STRUCTURAL OBSERVATION is the visual observation of the structural system by the structural observer as provided for in Section 107.9.2, for its general conformance to the approved plans and specifications, at significant construction stages and at completion of the structural system. Structural observation does not include or waive the responsibility for the structural inspections required by Section 107.1 or other sections of this code.

TERRACE is a relatively level step constructed in the face of a graded slope surface for drainage and maintenance purposes.

¹ *Guide to Investigation of Structural Failures, ASCE, 1986.*

² *The American Society of Civil Engineers (ASCE) Technical Council on Forensic Engineering.*

³ *Forensic Engineering, 2nd Edition, Kenneth L. Carper, Editor, 2001.*

⁴ *Robustness of Buildings in Structural Codes, Dimitris Diamantidis, 2009*

⁵ *Structural Failures in Buildings, ASCE, 1981.*

SECTION 103

CLASSIFICATION OF STRUCTURES

103.1 Nature of Occupancy

Buildings and other structures shall be classified, based on the nature of occupancy, according to Table 103-1 for purposes of applying wind and earthquake loads in Chapter 2. Each building or other structures shall be assigned to the highest applicable occupancy category or categories. Assignment of the same structure to multiple occupancy categories based on use and the type of loading condition being evaluated (e.g. wind or seismic) shall be permissible.

When buildings or other structures have multiple uses (occupancies), the relationship between the uses of various parts of the building or other structure and the independence of the structural system for those various parts shall be examined. The classification for each independent structural system of a multiple-use building or other structure shall be that of the highest usage group in any part of the building or other structure that is dependent on that basic structural system.

Table 103-1 Occupancy Category

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTION OF STRUCTURE
I Essential Facilities	Occupancies having surgery and emergency treatment areas, Fire and police stations, Garages and shelters for emergency vehicles and emergency aircraft, Structures and shelters in emergency preparedness centers, Aviation control towers, Structures and equipment in communication centers and other facilities required for emergency response, Facilities for standby power-generating equipment for Category I structures, Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category I, II or III, IV and V structures Public school buildings, Hospitals, Designated evacuation centers and Power and communication transmission lines.
II Hazardous Facilities	Occupancies and structures housing or supporting toxic or explosive chemicals or substances, Non-building structures storing, supporting or containing quantities of toxic or explosive substances.

Table 103-1 (cont'd) Occupancy Category

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTION OF STRUCTURE
III Special Occupancy Structures	Buildings with an assembly room with an occupant capacity of 1,000 or more, Educational buildings such as museums, libraries, auditorium with a capacity of 300 or more occupants, Buildings used for college or adult education with a capacity of 500 or more occupants, Institutional buildings with 50 or more incapacitated patients, but not included in Category I, Mental hospitals, sanitariums, jails, prisons and other buildings where personal liberties of inmates are similarly restrained, Churches, Mosques, and other Religion Facilities, All structures with an occupancy of 5,000 or more persons, Structures and equipment in power-generating stations, and other public utility facilities not included in Category I or Category II, and required for continued operation.
IV Standard Occupancy Structures	All structures housing occupancies or having functions not listed in Category I, II or III and Category V.
V Miscellaneous Structures	Private garages, carports, sheds and fences over 1.5m high.

SECTION 104 DESIGN REQUIREMENTS

104.1 Strength Requirement

Buildings, towers and other vertical structures and all portions thereof shall be designed and constructed to sustain, within the limitations specified in this code, all loads set forth in Chapter 2 and elsewhere in this code, combined in accordance with Section 203.

Design shall be in accordance with Strength Design, Load and Resistance Factor Design and Allowable Strength Design methods, as permitted by the applicable material chapters.

104.2 Serviceability Requirement

104.2.1 General

Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections, lateral drifts, vibration, or any other deformations that adversely affect the intended use and performance of buildings, towers and other vertical structures. The design shall also consider durability, resistance to exposure to weather or aggressive environment, crack control, and other conditions that affect the intended use and performance of buildings, towers and other vertical structures.

104.3 Analysis

Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics that take into account equilibrium, general stability, geometric compatibility and both short-term and long-term material properties. Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the added eccentricities expected to occur during their service life. Such analysis shall result in a system that provides a complete load path capable of transferring all loads and forces from their point of origin to the load-resisting elements. The analysis shall include, but not be limited to, the provisions of Sections 104.3.1 through 104.3.3.

104.3.1 Stability against Overturning

Every structure shall be designed to resist the overturning effects caused by the lateral forces specified with adequate Factor of Safety (FOS). See Section 206.6 for retaining walls, Section 207 for wind loading and Section 208 for earthquake loading.

104.3.2 Self-Straining Forces

Provisions shall be made for anticipated self-straining forces arising from differential settlement of foundations and from restrained dimensional changes due to temperature, moisture, shrinkage, heave, creep and similar effects.

104.3.3 Anchorage

Anchorage of the roof to walls and columns, and of walls and columns to foundations shall be provided and adequately detailed to resist the uplift and sliding forces that result from the application of the prescribed forces.

Concrete and masonry walls shall be anchored to all floors, roofs and other structural elements that provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in Chapter 2 but not less than the minimum forces in Section 206.4.

104.4 Foundation Investigation

Soil explorations shall be required for buildings, towers and other vertical structures falling under Categories I, II, III and IV in accordance with Table 103-1 or as required by the Building Official or if the site specific conditions make the foundation investigation necessary.

Detailed requirements for foundation investigations shall be in accordance with Chapter 3 of this code.

104.5 Design Review

The design calculations, drawings, specifications and other design-related documents for buildings, towers and other vertical structures with irregular configuration in Occupancy Categories I, II or III within Seismic Zone 4, structures under Alternative Systems in Section 101.4, and Undefined Structural Systems not listed in Table 208-11, shall be subject to a review by an independent recognized structural engineer or engineers to be employed by the owner in accordance with the ASEP Design Peer Review Guidelines. The structural engineer or structural engineers performing the review shall have comparable qualifications and experience as the structural engineer responsible for the design. The reviewer or reviewers shall obtain a professional waiver from the engineer-of-record who shall be expected to grant such waiver in keeping with ethical standards of the profession as adopted in ASEP guidelines for peer review (Appendix 1-A).

The design review shall, as a minimum, verify the general compliance with this code which shall include, but not be limited to, the review of the design load criteria, the design concept, mathematical model and techniques.

The following may also be verified, that there are no major errors in pertinent calculations, drawings and specifications and may also ensure that the structure as reviewed, meet minimum standards for safety, adequacy and acceptable standard design practice.

The engineer-of-record shall submit the plans and specifications, a signed and sealed statement by the structural engineer doing the review that the above review has been performed and that minimum standards have been met.

See Section 208.5.3.6.3.2 for design review requirements when nonlinear time-history analysis is used for earthquake design.

In keeping with the ethical standards of the profession, the reviewer or reviewers shall not supplant the engineer-of-record as engineer-of-record for the project. The design review shall not in any way transfer or diminish the responsibility of the engineer-of-record.

SECTION 105 POSTING AND INSTRUMENTATION

105.1 Posting of Live Loads

The live loads for which each floor or portion thereof of a commercial or industrial building has been designed shall have such design live loads conspicuously posted by the owner in that part of each story in which they apply, using durable metal signs. It shall not be allowed to remove or deface such notices. The occupant of the building shall be responsible for keeping the actual load below the allowable limits.

105.2 Earthquake-Recording Instrumentation

105.2.1 General

Unless waived by the Building Official, every building in Seismic Zone 4 over 50 m in height shall be provided with not less than three approved Earthquake Recording Instruments (ERI). The ERI shall be interconnected for common start and common timing. Please refer to "ASEP Guidelines and Implementing Rules on Earthquake Recording Instrumentation for Buildings (Appendix 1-B).

105.2.2 Location

The instruments shall be located in the basement, midportion, and near the top of the building. Each instrument shall be located so that access is maintained at all times and is unobstructed by room contents. A sign stating "MAINTAIN CLEAR ACCESS TO THIS INSTRUMENT" shall be posted in a conspicuous location.

105.2.3 Maintenance

Maintenance and service of the instruments shall be provided by the owner of the building, subject to the monitoring of the Building Official. Data produced by the instruments shall be made available to the Building Official or any authorized agency upon request.

105.2.4 Instrumentation of Selected Buildings

All owners of existing structures selected by the authorities having jurisdiction shall provide accessible space for the installation of appropriate earthquake-recording instruments, determined by a Structural Engineer.

SECTION 106 SPECIFICATIONS, DRAWINGS AND CALCULATIONS

106.1 General

Copies of design calculations, reports, plans, specifications and inspection program for all constructions shall bear the signature and seal of the engineer-of-record.

106.2 Specifications

The specifications shall contain information covering the material and construction requirements. The materials and construction requirements shall conform to the specifications referred to in Chapters 1 to 7 of this code.

106.3 Design Drawings

106.3.1 General

The design drawings shall be drawn to scale on durable paper or cloth using permanent ink and shall be of sufficient clarity to indicate the location, nature and extent of the work proposed. The drawings shall show a complete design with sizes, sections, relative locations and connection details of the various members. Floor levels, column centers and offsets shall be dimensioned. Where available and feasible, archive copies shall be maintained in durable medium such as compact disc (CD) and digital versatile disc (DVD).

106.3.2 Required Information

The design drawings shall contain, but shall not be limited to the general information listed in Section 106.3.2.1 and material specific information listed in Sections 106.3.2.2 and 106.3.2.3, as applicable.

106.3.2.1 General Information

1. Name and date of issue of building code and supplements, if any, to which the design conforms.
2. Strengths or designations of materials to be used.
3. Design strengths of underlying soil or rock. The soil or rock profile, when available, shall be provided.
4. Live loads and other loads used in design and clearly indicated in the floor plans.

5. Seismic design basis including the total base shear coefficient; a description of the lateral load resisting system; and the fundamental natural period in the design in each direction under consideration.
6. Provisions for dimensional changes resulting from creep, shrinkage, heave and temperature.
7. Camber of trusses, beams and girders, if required.
8. Explanation or definition of symbols and abbreviations used in the drawings.
9. Engineer-of-Record's professional license number and expiration date of the current Professional Regulation Commission registration (PRC).

106.3.2.2 Structural Concrete

1. Specified compressive strength (f'_c) of concrete at stated ages or stages of construction for which each part of structure designed. The 28-day compressive strength (f'_c) shall be the basis of design in service condition.
2. Anchorage embedment lengths or cut-off points of steel reinforcement and location and length of lap splices.
3. Type and location of welded splices and mechanical connections of reinforcement.
4. Magnitude and location of prestressing forces including prestressed cable layout
5. Minimum concrete compressive strength (f_{ci}) at time of post-tensioning.
6. Stressing sequence for post-tensioned tendons.
7. Details and location of all contraction or isolation joints specified for plain concrete in Chapter 4.
8. Statement if concrete slab is designed as a structural diaphragm, as specified in Sections 421.9.4 and 421.9.5.

SECTION 107 STRUCTURAL INSPECTIONS, TESTS AND STRUCTURAL OBSERVATIONS

107.1 General

All construction or work for which a permit is required shall be subject to inspection throughout the various work stages. One or more structural inspectors who are registered civil engineers with experience in structural construction, who shall undertake competent inspection during construction on the types of work listed under Section 107.5, shall be employed by the owner or the engineer-of-record acting as the owner's agent.

Exception:

The Building Official may waive the requirement for the employment of a structural inspector if the construction is of a minor nature.

In addition to structural inspections, structural observations shall be performed when required by Section 107.9.

107.2 Definitions

See Section 102 for definitions.

107.3 Structural Inspector

107.3.1 Qualifications

The structural inspector shall be a registered civil engineer who shall demonstrate competence for inspection of the particular type of construction or operation requiring structural inspection.

107.3.2 Duties and Responsibilities

The structural inspector shall observe the work assigned for conformance to the approved design drawings and specifications. Any discrepancy observed shall be brought to the immediate attention of the constructor for correction, then, if uncorrected, to the owner, engineer-of-record and/or to the Building Official.

The structural inspector shall verify that the as-built drawings (see Section 106.5) pertaining to the work assigned reflect the condition as constructed.

The structural inspector shall also submit a final report duly signed and sealed stating whether the work requiring structural inspection was, to the best of the inspector's knowledge, in conformance to the approved plans and specifications and the applicable workmanship provisions of this code.

107.4 Inspection Program

The structural inspector shall prepare an appropriate testing and inspection program that shall be submitted to the owner, engineer-of-record and/or to the Building Official. He shall designate the portions of the work that requires structural inspections.

When structural observation is required by Section 107.9, the inspection program shall describe the stages of construction at which structural observation is to occur.

The inspection program shall include samples of inspection reports and provide time limits for submission of reports.

107.5 Types of Work for Inspection

Except as provided in Section 107.1, the types of work listed below shall be inspected by a structural inspector.

107.5.1 Concrete

During the taking of test specimens and placing of concrete. See Section 107.5.12 for shotcrete.

Exceptions:

1. Concrete for foundations of residential buildings accommodating 10 or fewer persons, or buildings falling under Category V of Table 103-1, provided the Building Official finds that a structural hazard does not exist.
2. For foundation concrete, other than cast-in-place drilled piles or caissons, where the structural design is based on f'_c not greater than 17 MPa.
3. Non-structural slabs on grade, including prestressed slabs on grade when effective prestress in concrete is less than 10 MPa.
4. Site work concrete fully supported on earth and concrete where no special hazard exists.

107.5.2 Bolts Installed in Concrete

Prior to and during the placement of concrete around bolts when stress increases permitted by Section 426 are utilized.

107.5.3 Special Moment-Resisting Concrete Frame

For special moment-resisting concrete frame design seismic load in structures within Seismic Zone 4, the structural inspector shall provide reports to the engineer-of-record and shall provide continuous inspection of the placement of the reinforcement and concrete.

107.5.4 Reinforcing Steel and Prestressing Steel Tendons

107.5.4.1 During all stressing and grouting of tendons in prestressed concrete.

107.5.4.2 During placing of reinforcing steel and prestressing tendons for all concrete required to have structural inspection by Section 107.5.1.

Exception:

The structural inspector need not be present continuously during placing of reinforcing steel and prestressing tendons, provided the structural inspector has inspected for conformance to the approved plans prior to the closing of forms or the delivery of concrete to the jobsite.

107.5.5 Structural Welding

107.5.5.1 General

During the welding of any member or connection that is designed to resist loads and forces required by this code.

Exceptions:

1. Welding done in an approved fabricator's shop in accordance with Section 107.6.
2. The structural inspector need not be continuously present during welding of the following items, provided the materials, qualifications of welding procedures and welders are verified prior to the start of work; periodic inspections are made of work in progress; and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding:
 - a) Single-pass fillet welds not exceeding 8 mm in size.
 - b) Floor and roof deck welding.
 - c) Welded studs when used for structural diaphragm or composite systems.
 - d) Welded sheet steel for cold-formed steel framing members such as studs and joists.
 - e) Welding of stairs and railing systems.

107.5.5.2 Special Moment-Resisting Steel Frames

During the non-destructive testing (NDT) of welds specified in Section 107.8 of this code, the use of certified welders shall be required for welding structural steel connections for this type of frame. Critical joint connections shall be subjected to non-destructive testing using certified NDT technicians.

107.5.5.3 Welding of Reinforcing Steel

During the non-destructive testing of welds.

107.5.6 High-Strength Bolts

The inspection of high-strength A325 and A490 bolts shall be in accordance with approved internationally recognized standards and the requirements of this section. While the work is in progress, the structural inspector shall determine that the requirements for bolts, nuts, washers and paint; bolted parts; and installation and tightening in such standards are met. Such inspections may be performed on a periodic basis as defined in Section 107.

The structural inspector shall observe the calibration procedures when such procedures are required by the plans or specifications. He shall monitor the installation of bolts to determine that all layers of connected materials have been drawn together and that the selected procedure is properly used to tighten all bolts.

107.5.7 Structural Masonry

107.5.7.1 For masonry, other than fully grouted open-end hollow-unit masonry, during preparation and taking of any required prisms or test specimens, placing of all masonry units, placement of reinforcement, inspection of grout space, immediately prior to closing of cleanouts, and during all grouting operations.

Exception:

For hollow-unit masonry where the f_m is no more than 10 MPa for concrete units or 18 MPa for clay units, structural inspection may be performed as required for fully grouted open-end hollow-unit masonry specified in Section 107.5.7.2.

107.5.7.2 For fully grouted open-end hollow-unit masonry during preparation and taking of any required prisms or test specimens, at the start of laying units, after the placement of reinforcing steel, grout space prior to each grouting operation, and during all grouting operations.

Exception:

Structural inspection as required in Sections 107.5.7.1 and 107.5.7.2 need not be provided when design stresses have been adjusted as specified in Chapter 7 to permit noncontinuous inspection.

107.5.8 Reinforced Gypsum Concrete

When cast-in-place Class B gypsum concrete is being mixed and placed.

107.5.9 Insulating Concrete Fill

During the application of insulating concrete fill when used as part of a structural system.

Exception:

The structural inspections may be limited to an initial inspection to check the deck surface and placement of reinforcing steel. The structural inspector shall monitor the preparation of compression test specimens during this initial inspection

107.5.10 Spray-Applied Fire-Resistive Materials

During the application of spray-applied fire-resistive materials.

107.5.11 Piling, Drilled Piers and Caissons

During driving and load testing of piles and construction of cast-in-place drilled piles or caissons. See Sections 107.5.1 and 107.5.4 for concrete and reinforcing steel inspection.

107.5.12 Shotcrete

During the taking of test specimens and placing of all shotcrete.

Exception:

Shotcrete work fully supported on earth, minor repairs and when, in the opinion of the Building Official, no special hazard exists.

107.5.13 Special Grading, Excavation and Filling

During earthwork excavations, grading and filling operations inspection to satisfy requirements of Chapter 3 and Section 109.5.

107.5.14 Special Cases

Work that, in the opinion of the structural engineer, involves unusual hazards or conditions.

107.5.15 Non-Destructive Testing

In-situ non-destructive testing program, in addition to the requirements of Section 107.8 that in the opinion of the structural engineer may supplement or replace conventional tests on concrete or other materials and assemblies.

107.6 Approved Fabricators

Structural inspections required by this section and elsewhere in this code are not required where the work is done on the premises of a fabricator approved by the structural engineer to perform such work without structural inspection. The approved fabricator shall submit a certificate of compliance that the work was performed in accordance with the approved plans and specifications to the Building Official and to the engineer or architect-of-record. The approved fabricator's qualifications shall be contingent on compliance with the following:

1. The fabricator has developed and submitted a detailed fabrication procedural manual reflecting key quality control procedures that will provide a basis for inspection control of workmanship and the fabricator plant.

2. Verification of the fabricator's quality control capabilities, plant and personnel as outlined in the fabrication procedural manual shall be by an approved inspection or quality control agency.
3. Periodic plant inspections shall be conducted by an approved inspection or quality control agency to monitor the effectiveness of the quality control program.

107.7 Prefabricated Construction**107.7.1 General****107.7.1.1 Purpose**

The purpose of this section is to regulate materials and establish methods of safe construction where any structure or portion thereof is wholly or partially prefabricated.

107.7.1.2 Scope

Unless otherwise specifically stated in this section, all prefabricated construction and all materials used therein shall conform to all the requirements of this code.

107.7.1.3 Definition

See Section 102 for Definitions.

107.7.2 Tests of Materials

Every approval of a material not specifically mentioned in this code shall incorporate as a proviso the kind and number of tests to be made during prefabrication.

107.7.3 Tests of Assemblies

The Building Official may require special tests to be made on assemblies to determine their structural adequacy, durability and weather resistance.

107.7.4 Connections

Every device used to connect prefabricated assemblies shall be designed as required by this code and shall be capable of developing the strength of the largest member connected, except in the case of members forming part of a structural frame designed as specified in Chapter 2. Connections shall be capable of withstanding uplift forces as specified in Chapter 2.

107.7.5 Pipes and Conduits

In structural design, due allowance shall be made for any material to be removed or displaced for the installation of pipes, conduits or other equipment.

107.7.6 Certificate and Inspection**107.7.6.1 Materials**

Materials and the assembly thereof shall be inspected to determine compliance with this code. Every material shall be graded, marked or labeled where required elsewhere in this code.

107.7.6.2 Certificate

A certificate of acceptance shall be furnished with every prefabricated assembly, except where the assembly is readily accessible to inspection at the site. The certificate of acceptance shall certify that the assembly in question has been inspected and meets all the requirements of this code.

107.7.6.3 Certifying Agency

To be acceptable under this code, every certificate of approval shall be made by a nationally or internationally recognized certifying body or agency.

107.7.6.4 Field Erection

Placement of prefabricated assemblies at the building site shall be inspected to determine compliance with this code.

107.7.6.5 Continuous Inspection

If continuous inspection is required for certain materials where construction takes place on the site, it shall also be required where the same materials are used in prefabricated construction.

Exception:

Continuous inspection will not be required during prefabrication if the approved agency certifies to the construction and furnishes evidence of compliance.

107.8 Non-Destructive Testing**107.8.1 General**

In Seismic Zone 4, welded, fully-restrained connections between the primary members of special moment-resisting frames shall be tested by nondestructive methods performed by certified NDT technicians for compliance with approved standards and job specifications. This testing shall be a part of the structural inspection requirements of Section 107.5. A program for this testing shall be established by the person responsible for structural design and as shown on plans and specifications.

107.8.2 Testing Program

As a minimum, the testing program shall include the following:

107.8.2.1 All complete penetration groove welds contained in joints and splices shall be tested 100 percent either by ultrasonic testing or by radiography.*Exceptions:*

1. *When approved, the non-destructive testing rate for an individual welder or welding operator may be reduced to 25 percent, provided the reject rate is demonstrated to be 5 percent or less of the welds tested for the welder or welding operator. A sampling of at least 40 completed welds for a job shall be made for such reduction evaluation. Reject rate is defined as the number of welds containing rejectable defects divided by the number of welds completed. For evaluating the reject rate of continuous welds over 900 mm in length where the effective throat thickness is 25 mm or less, each 300 mm increment or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 900 mm in length where the effective throat thickness is greater than 25 mm, each 150 mm of length or fraction thereof shall be considered one weld.*
2. *For complete penetration groove welds on materials less than 8 mm thick, non-destructive testing is not required; for this welding, continuous inspection is required.*
3. *When approved by the Building Official and outlined in the project plans and specifications, this non-destructive ultrasonic testing may be performed in the shop of an approved fabricator utilizing qualified test techniques in the employment of the fabricator.*

107.8.2.2 Partial penetration groove welds when used in column splices shall be tested either by ultrasonic testing or radiography when required by the plans and specifications. For partial penetration groove welds when used in column splices, with an effective throat less than 20 mm thick, nondestructive testing is not required; for this welding, continuous structural inspection is required.

107.8.2.3 Base metal thicker than 40 mm, when subjected to through-thickness weld shrinkage strains, shall be ultrasonically inspected for discontinuities directly behind such welds after joint completion.

Any material discontinuities shall be accepted or rejected on the basis of the defect rating in accordance with the (larger reflector) criteria of approved national standards.

107.8.3 Others

The structural engineer may accept or require in place non-destructive testing of concrete or other materials and assemblies to supplement or replace conventional tests.

107.9 Structural Observation

107.9.1 General

Structural observation shall be provided in Seismic Zone 4 when one of the following conditions exists:

1. The structure is defined in Table 103-1 as Occupancy Category I, II, III and IV.;
2. The structure is in Seismic Zone 4, N_a as set forth in Table 208-4 is greater than 1.0, and a lateral design is required for the entire structure;
3. When so designated by the structural engineer, or
4. When such observation is specifically required by the Building Official.

107.9.2 Structural Observer

The owner shall employ the engineer-of-record or another civil engineer to perform structural observation as defined in Section 107.

Observed deficiencies shall be reported in writing to the owner's representative, structural inspector, constructor and the Building Official. If not resolved, the structural observer shall submit to the Building Official a written statement duly signed and sealed, identifying any deficiency.

107.9.3 Construction Stages for Observations

The structural observations shall be performed at the construction stages prescribed by the inspection program prepared as required by Section 107.4.

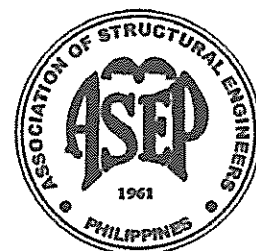
It shall be the duty of the engineer-in-charge of construction, as authorized in the Building Permit, to notify the structural observer that the described construction stages have been reached, and to provide access to and means for observing the components of the structural system.

APPENDIX 1-A

RECOMMENDED GUIDELINES ON STRUCTURAL DESIGN PEER REVIEW OF STRUCTURES 2015

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APPENDIX 1-A RECOMMENDED GUIDELINES ON STRUCTURAL DESIGN PEER REVIEW OF STRUCTURES

About this Guidelines

Recommended Guidelines on Structural Design Peer Review of Structures 2015
Published by Association of Structural Engineers of the Philippines, Inc.

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About ASEP

The Association of Structural Engineers of the Philippines, Inc. (ASEP) is the recognized organization of Structural Engineers of the Philippines. Established in 1961, ASEP has been in existence for more than 50 solid years.



Print History
2000
2015

CONTENTS

ABBREVIATIONS.....	I-A6
INTRODUCTION	I-A7
BACKGROUND.....	I-A8
OBJECTIVES OF THE DESIGN PEER REVIEW.....	I-A9
APPLICATION OF ASEP PEER REVIEW GUIDELINES	I-A9
EXPECTED RESULTS OF DESIGN REVIEW:.....	I-A10
STRUCTURES TO BE REVIEWED	I-A10
REVIEWER'S QUALIFICATION.....	I-A11
SCOPE OF REVIEW.....	I-A11
INFORMATION TO BE FURNISHED TO PEER REVIEWER.....	I-A11
ITEMS TO BE REVIEWED	I-A 12
METHODOLOGY AND DETAILS OF REVIEW	I-A15
<i>Design Basis Review</i>	I-A15
<i>Foundation Review</i>	I-A16
<i>Pre-Tender Design Review</i>	I-A16
MINIMUM REPORT REQUIREMENTS.....	I-A16
<i>Content</i>	I-A16
<i>Terms of Review Procedure and Methodology to be Used</i>	I-A17
<i>Language to be Used</i>	I-A17
<i>Mark-up and Comments</i>	I-A17
<i>Examples of Reviewer's Comments/Wordings</i>	I-A18
REFERENCES	I-A18

Abbreviations

A&D	Analysis and Design
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ASEP	Association of Structural Engineers of the Philippines, Inc.
BIM	Building Information Model
CE	Civil Engineer
CTBUH	Council on Tall Buildings and Urban Habitat
DPWH	Department of Public Works and Highways
EOR	Engineer-of-Record
IAI	International Alliance for Interoperability
IBC	International Building Code
IFC	Industry Foundation Classes
NSCP	National Structural Code of the Philippines
PAGASA	Philippine Atmospheric, Geophysical & Astronomical Services Administration
PHIVOLCS	Philippine Institute of Volcanology and Seismology
PR	Peer Reviewer
PRC	Professional Regulations Commission
SE	Structural Engineer
SEC	Security and Exchange Commissions
UBC	Uniform Building Code

Introduction

Design review is incorporated in most building codes to provide the means for professional discussion and evaluation of structural design of projects. Thus, these reviews are the eye openers for the resolution of problems encountered before a critical phase of the construction project. Design review truly enhances the ideas for public safety overall and quality assurance. Furthermore, it disseminates innovation through sharing of information.

Earthquake for instance is a phenomenon that man has been trying to study for centuries but up to present time is still unpredictable. We, as structural engineers, are faced with the greatest challenge of formulating procedures on how to lessen if not eliminate destruction and casualties due to this. We want to make sure that the intent of our design is carefully followed and carried out in the most professional manner. The burden of setting up and observing rules on how to achieve what has been planned rest upon our shoulders. Design review can be a valuable tool faced with this challenge.

This document establishes the guidelines for peer review. Since protecting lives and properties are the paramount goals of the Association of Structural Engineers of the Philippines (ASEP), the only way perhaps to realise these goals is to establish ground rules for all our practicing civil engineers, structural engineers and consultants to follow strictly the Code provisions and standards parameters.

It is essential to good engineering practice to conduct independent peer review to achieve a concept of structural system and design tolerant to the crudeness in seismological predictions. The independent review of structures shall be deemed as the means to promote life safety, achieve excellence in structural design and front of quality, improvement/advancement and dissemination of structural engineering knowledge in the country.

Background

To accomplish the objectives of ASEP, the Board of Directors for 1999-2000 has continued the program of the ASEP Board of Directors for 1998-1999 by creating several committees as shown below. These objectives, as stated in its by-laws, shall be the protection of the public welfare and the welfare of its constituents through the:

- Maintenance of highly ethical and professional standards in the practice of engineering
- Advancement of structural engineering knowledge;
- Promotion of good public and private clientele relationships, development of fellowships among CE and SE and encouragement of professional relations with other allied technical and scientific organizations.

These objectives are focused on these three major areas:

- Codes and Standards
- Fellowships and Linkages
- Technical Advancement

One of the committees created for the Codes and Standards is the Committee on Design Peer Review. The National Structural Code of the Philippines (NSCP) 1992 Edition touches on independent design review under the section "A Design and Construction Review", which defines the structures required for the review considering seismic zones and occupancy categories. However, the scope, procedures and documentation of the review process are not mentioned. Thus, this paper will include guidelines on the implementation of the design peer review.

The same committee was revived by the President of the Board of Directors for 2009-2010, Adam C. Abinales, from the point of view of engineering practitioners, to improve and expand the guidelines to incorporate additional parameters and ethical rules as well as enhance the practice of peer review. The committee's activities have continued under the administration of the following ASEP Presidents:

- Anthony Vladimir Pimentel (2010-2011)
- Vinci Nicholas R. Villasenor (2011-2012)
- Miriam L. Tamayo (2013)
- Carlos M. Villaraza (2015)

The Committee on Design Peer Review is composed of the following:

Chairman

Ernesto F. Cruz

Co-Chairman

Gabriel Ursus L. Eusebio

Members

Alden C. Ong
 Marie Christine G. Danao
 Edmondo D. San Jose

Objectives of the Design Peer Review

The current trend in the local construction industry is the development of many high-rise buildings. On account of this, it is the objective of this peer review to improve section 104.5 of NSCP 2010, to ensure the aim for life safety, to observe economy in design and to protect the investment of clients.

The Peer Review aims to carry out positive results in the following areas:

- To comply structural engineering design, drawings and specification with the minimum requirements of NSCP and other acceptable established codes and standards;
- To maintain the quality of projects;
- To improve and maintain the high standards in the practice of structural engineering;
- To promote exchange of information and innovative ideas between the designers and reviewers;
- To inform the Owner-Client on the benefits of this exercise and any possible cost implications resulting from the review;
- To define implementing matrix of all structures subject to practical independent review; and
- To promote professional ethics in the conduct of independent or peer review.

Application of ASEP Peer Review Guidelines

These ASEP guidelines are intended specifically for the mandatory conduct of a Design Review as per the National Structural Code of the Philippines (NSCP Volume 1, 2010 Edition).

As stipulated in NSCP Section 104.5, Design Review is required for the following:

1. Structures with irregular configuration in Occupancy Categories I (Essential Facilities), II (Hazardous Facilities) or III (Special Occupancy Structures) in Zone 4;
2. Structures under Alternative Systems in Section 101.4; and,
3. Undefined Structural Systems (those not listed in Table 208-11).

For structures covered by the mandatory Design Review, all related works shall be deemed as included in the Engineer-of-Record's scope of works, unless explicitly excluded in his work agreement.

For structures not included above but which are to be subjected to a Design Review as an additional requirement by the Owner, the coverage, extent, and procedures shall be as mutually agreed upon by the Owner/Peer Reviewer, and the Engineer-of-Record (EOR) and may not be as recommended in these Guidelines. Additionally, since works connected or related to such Design Review are not covered by the basic structural services of the EOR, these shall be subject to a separate scope and compensation for the EOR.

Expected Results of Design Review:

- As professionals, independent design reviewers and EOR shall not engage in unfair practices. Both shall observe fairness and professionalism in the practice of independent review. This shall not by any means be a channel to conduct criticism nor be a means to discredit the reviewer or the EOR, and disenfranchise them of the contract service they are awarded.
- There will be good understanding of the structures and relationships between the Owner-Client and the structural engineering community resulting to enhanced programs of future developments and projects.
- There will be good relationships between designers and reviewers by improving the design through constructive reporting.
- The review will be conducted smoothly in the light of fairness and professionalism, without unfair practice and criticism to neither discredit nor disenfranchise any of the reviewer or EOR.
- The review will bring assurance to the Owner-Client of compliance to codes and standards, assurance of better engineering of the proposed structure, the improvement in design and safety as well as improvement in construction implementation and program, elimination of unsafe design and possible work delays from unwanted and costly repairs, among others.

Structures to be Reviewed

Structures to be reviewed shall consist of all proposed new structures and addition to structures which shall be deemed crucial to life safety and/or health of the public and peace if such structures or buildings would incur damage or failure or both.

The structures to be reviewed shall be as follows:

1. All structures more than 75-meter high (whichever is higher) from the exterior ground level.
2. Buildings, towers and other vertical structures with irregularity in configuration (vertical and horizontal irregularity) under occupancy Category I, II, and III (as per section 103.1 NSCP VI edition) within the seismic zone 4.
3. Structures designed under alternative system (as per section 101.4 NSCP VI Edition) that intends to use other structural materials, design approach and construction methodology not prescribed by the latest existing structural Code (NSCP VI Edition, 2010) or by other recognized international codes and standards.
4. Buildings, towers and other structures with undefined structural system not listed in Table 208-11 of NSCP VI Edition.
5. Essential facilities such as hospitals fire & police stations, emergency vehicle and equipment shelters and garages, structures and equipment in communication center, aviation control towers, private and public school buildings, water supported structures and designated evacuation center, also buildings and structures for national defense.
6. Hazardous Facilities and the like structures housing, supporting or containing sufficient quantities of toxic or explosive substances dangerous to the safety of the general public if released due to damage or excessive deformation.

Reviewer's Qualification

The independent PR shall be nominated by the Owner-Client. The independent PR shall not be the design EOR or engineer appointed by Builder/Contractor. In the case of Turnkey or Design and Build projects whose design is initiated by the Contractor or Developer, the Contractor at his own expense shall appoint an independent recognized structural engineer to conduct the services of design peer review.

The independent PR shall have the following qualifications:

1. Civil engineer registered with the Professional Regulation Commission of the Philippines with more than 20 years of related structural engineering experience similar to the structure to be reviewed.
2. He must be a REGULAR ASEP Member in good standing.
3. Structural Engineers with comparable qualification and experience as the EOR responsible for the design (as per latest NSCP).
4. Knowledgeable in current design software, tools, and other acceptable current computer programs.
5. Have competitive knowledge or experience in actual structural construction.

Scope of Review

The PR must review all items agreed to be reviewed with the Owner-Client and EOR per relevant/recommended items listed in this Guide. The PR shall refer regularly to check for completeness of the review per applicable items listed in these guidelines. The quantity of elements to be reviewed shall be in accordance with the second paragraph of the subsection Methodology of Review below.

Information to be Furnished to Peer Reviewer

The review documents should be checked for completeness and timeliness of the design documents submitted per relevant items recommended in this guide. The PR should assess the review documents received and report immediately to the Owner-Client and/or his duly appointed representative for the following:

- If any of the design documents submitted are not sufficient for him to proceed with the review such that an entire document is missing, for example the design criteria document is not included and the drawings do not reflect the design parameters/information completely; or
- The documents given and received may enable him to start and work immediately but the PR have to stop soon for some items of works as some documents are given as partial only; or
- The documents given and submitted are irrelevant to the project; or
- The documents received are of poor quality such as illegible, faintly printed, blurred, torn, and or unacceptably dirty or laced with hazardous materials.

- The PR shall also report if the items received were not delivered in good condition that may not enable him to proceed at all; e.g. the documents are wet due to improper handling, incomplete or inadequate protection from packaging materials, among others.

The following items are to be furnished by the Owner-Client as applicable:

- ☐ Printed copies or PDF/DWFX format of complete set of architectural and structural drawings;
- ☐ General building narrative (number of stories, gross building area, estimated construction cost, unique features, among others);
- ☐ Geotechnical engineering report;
- ☐ Wind Tunnel Test report (if any);
- ☐ Site-specific spectra and ground-motion time histories (if any);
- ☐ Major equipment or special loadings;
- ☐ Existing building drawings/data if impacted by or impacting the threshold structure;
- ☐ Analysis models including User's Guide of software used by EOR (e.g. STAAD, ETABS, SAP, SAFE and midasGen). It is recommended to include also interoperable files such as .SET, .ANL, .S2K and .F2K to facilitate conversion of data.;
- ☐ 3D model/BIM¹ file or *.ifc² file (if any);
- ☐ Design basis;
- ☐ Design criteria;
- ☐ Structural systems design narrative (including wind and seismic design parameters);
- ☐ Structural elements design calculations; and
- ☐ Structural specifications.

Items to be Reviewed

The PR may include as appropriate/applicable any or all of the following:

Table 1: Checklist of Items to be Reviewed

Item	Specific Design Checks to be Carried Out
Design Basis/Criteria	Minimum loadings as set out in the code.
	Prevailing site conditions and assumptions in design analysis.
	Materials used in the design and specifications.
	Reference to any assumed loadings, construction methods, A&D.
	Description of the operational language and/or algorithms, capability and source of the software used, including the proof of good comparison with results of known and accepted method of analysis.
	Seismic design parameters and base shear.
	Number of mass participation for dynamic analysis.

¹ Building Information Model (BIM) is a digital representation of physical and functional characteristics of a structure. As such it serves as a shared knowledge resource for information about a structure forming a reliable basis for decisions during its life cycle from inception onward.

² Industry Foundation Classes (.ifc) – A file format developed by the IAI. IFC provides an interoperability solution between IFC-compatible software applications in the construction and facilities management industry. The format has established, international standards to import and export building objects and their properties.

Item	Specific Design Checks to be Carried Out
Design Methods, Standards and Specifications	Wind loadings design parameters and comfort criteria.
	Appropriateness to the Client's technical brief and functional requirements.
	Conformance to the governing codes used in the analysis and design.
Analysis models	Input and output data including geometry, material constants, properties, loadings, assignments and parameters used in software used.
Foundation Loads	Appropriate values of dead, live, wind and seismic loads used.
	Column loads have been appropriately computed and compared to results of analysis model.
	Effects of wind and notional loads on the building or structure have been checked.
Piles	Pile capacities have been designed for compressive axial load by applicable skin friction and end bearing capacities.
	Review if the type of pile reactions used in the analysis models are appropriate/applicable.
	Piles were checked if required/applicable for combined buckling.
	Piles have been designed for lateral loads and bending moment.
	Pile joints have been designed for anchorage and embedment length of reinforcements, for concrete pile.
	Piles have been designed for uplift.
	Socketing has been designed for piles with short penetration depths.
	Piles have been designed for negative skin friction.
Isolated Pads/ Combined Footings/ Tied Footings	Checked for punching shear and bending moments.
Raft	Appropriate allowable bearing capacity of soil has been assumed in design.
	Appropriate modulus of sub-grade reaction of the soil has been assumed in design.
	Appropriate model used for structural analysis of the raft.
	The raft has been designed to resist punching shear from columns.
	The building or structure has been designed to cater for probable differential and total settlement.
Lateral Load Resisting Framing Systems as assumed in the Design Basis/ Criteria	The presence in the structural framing of any plan and/or vertical irregularities mentioned in NSCP or governing codes.
	Limitations of lateral load resisting framing systems by NSCP, or by the Owner-Client preferred code and standards and or from any prevailing local ordinance and regulations in the vicinity of the proposed structure.
	Details of seismic-resistant concrete structure were checked.
Slender Columns	Effective height has been computed according to code.
	Bending moment about minor axis has been designed for.
	Additional bending moment due to slenderness has been designed for.
	Biaxial bending moment has been designed for.
Columns supporting transfer beams	Requirement for ductility such as strong-column weak beam is provided or complies with the code.
	Designed for bending moment due to frame action including effects of special load combination per code.

Item	Specific Design Checks to be Carried Out
Columns supporting long span beams	Designed for bending moment due to frame action.
Columns supporting cantilever beams	Designed for bending moment due to frame action.
Columns in a two column frame system	Designed for horizontal load and moment acting on columns due to arched or pitched roof.
	Designed for bending moment and shear at the column base including connections.
	Designed for bending moment due to frame action.
Cantilever beams	Cantilever support has been designed to resist bending moment and shear including minimum uplift loads from wind and seismic loads.
	Designed for lateral stability of beam.
	Designed to meet allowable span depth ratio; else deflection against allowable limit per code including long-term effects.
Long span beams	Torsional rigidity of beam has been checked.
	Designed for lateral restraint of beams.
	Designed for support and member connections.
	Designed to meet allowable span depth ratio; else deflection against allowable limit per code including long-term effects.
Transfer beams	Designed for torsional capacity.
	Designed for shear capacity.
	Designed for all relevant upper floor loads on the beam including effects of special load combinations per code.
	Designed for lateral restraint of beam.
Flat slabs/plates	Appropriate model used for analysis.
	Span/depth ratio of slab has been checked.
	Adequacy of top and bottom reinforcement throughout slab panel has been checked.
	Designed to resist punching shear from columns.
	Openings in slabs, especially near columns, have been designed for.
	Torsional rigidity at slab edges has been checked.
	Effects of construction loads have been checked.
Engineering drawings	Clarity and consistency with the design intent of the architect and consultants, design bases and calculations, site surveys and investigations.
	Complete sections and details.
	Consistency with and conformance to the specifications.
	Consistency of the revisions and/or amendments to the design basis and criteria and their compliance with the design intent and Client requirement.
Structural calculations	Consistency of design loading with the criteria and the equipment supplier/vendors data, finishes, plus the possible construction method requirements, effects of foreseen temporary works and activities during construction, among others.
	Usage of correct wind/seismic load parameters for analysis and design with regards to the structures lateral load resisting framing system, seismic zone, material type and structural framing plan or vertical irregularity.
	Seismic load analysis if requiring P-delta effects and/or dynamic method as to height limitations and irregularities.
	Load combinations and special load combinations as required and prescribed by the code.
	Structural geometric model for completeness of the structures vertical load carrying elements and for consistency with the basis and criteria.

Item	Specific Design Checks to be Carried Out
	Member and element checks such as minimum reinforcements and details, strength requirements, slenderness effects, joints forces checks and connection requirements.
Structural deformation and displacement checks	Drift limitation of the structures (service and ultimate state). Size of movement joints or expansion joints. Girder and secondary beam deflections. Deformation compatibility on non-lateral load resisting elements.
Stability/factors of safety check	Overall and local structural stability against overturning and sliding. Compliance with factors of safety for miscellaneous requirements by clients.
Earth retaining structures	Structure has been designed to resist overturning, sliding and bearing capacity failure. Structure has been designed to resist slip circle failure. Structure has been designed for water pressure acting on it. Adequate surcharge load has been taken into account in design. Embedment into ground for stability has been designed for in cantilevered structures.
Frame	Check for type limitations if dual system, SMRF, etc. against height and materials. Checks for plan and vertical irregularities.

Methodology and Details of Review

The PR should agree with the Owner-Client and the EOR on the methodology of review. The review shall cover for completeness and timeliness of the design documents submitted per relevant items listed in this Guide.

The PR should assess the review documents with regards to the agreed number of elements to be checked with the Owner-Client and/or his representative, if at random, selected or full review of the structure and any limited procedure.

Review may be agreed also for each phase or entirely on the final detailed design phase of the structure for review. While a final detailed design review is basically economical, a phased review from the beginning may be better in order to avoid the errors from the beginning and save also valuable time in re-work.

Design Basis Review

The PR should do the following:

1. Review design criteria to verify compliance with the building code;
2. Assess assumptions made by the EOR; and
3. Review the proposed frame system/s and load path for vertical load-carrying elements.

Foundation Review

The PR should do the following:

1. Establishes foundation loads via independent analysis. Alternatively, obtain foundation loads from EOR contingent upon subsequent verification. The PR should obtain soil design parameters from geotechnical engineering report.
2. Perform independent analysis of representative foundation elements including spread footings, pile caps, foundation walls, grade beams and piles, among others. Review of foundation elements is recommended, depending on the relative nature or complexity of the project.
3. Review specification sections pertaining to foundation system including earthwork, piles, concrete work, among others.
4. Review performance criteria for contractor-designed components such as slope protection systems, mini piles, tie-down anchors, among others.

Pre-Tender Design Review

The PR should carry out the following:

1. Review structural framing connections which are part of the primary system including shear connections, braced frame connections, moment-resisting connections, among others. When connections are not detailed on the design drawings, verify adequacy of the cited connection design loads/procedures.
2. Perform general review of design to evaluate presence of any conditions which might precipitate instability or structural overstress.
3. Review specification sections pertaining to Primary Structural Support System.
4. Review performance criteria for contractor-designed components such as pre-cast concrete elements, shear connections, braced frame connections, moment-resisting connections, cold-formed metal framing components (primary framing components, not cladding), pre-engineered metal building systems, among others.

Minimum Report Requirements

Prior to the issuance of the final peer review report, the PR is encouraged to exchange review comments with the EOR in the presence of the Owner and/or his representative in order to resolve as many issues as possible.

Content

The following items shall be included in the final peer review report:

1. List of the documents on which the review was based;
2. Building Codes and Standards on which the peer review was based;
3. Methodology and assumptions of the review;
4. List of software/analysis tools used with descriptive statements about software, tools and other computer programs used in the review;
5. Items to be subsequently reviewed by others (e.g. contractor-designed items);
6. Exclusions/limitations (e.g. peer review was limited to primary structural support systems);
7. Outstanding items/unresolved issues; and

8. Results, findings, conclusions and recommendations of the review.
9. The final peer review report shall be addressed to the Owner-Client/representative and the EOR. Upon completion of the review, the PR shall issue a certificate stating that the peer review has been successfully completed

Terms of Review Procedure and Methodology to be Used

The review analysis and design criteria must meet the requirement of the Owner-Client as defined in his design brief including any applicable item in the Terms of Reference which form part of their agreement with the EOR.

Preferably, the PR shall use the same design criteria and standards specified by the EOR. Deviations from the said criteria and standards must be done only with the permission of the EOR.

Software to be used in the review should preferably be the same software used by the EOR (e.g. the same editions or versions). The difference of versions should be agreed upon but a difference of one level may be considered acceptable unless the more recent versions employ a different analysis procedures or features that are almost entirely different to the EOR's software procedures or features.

Language to be Used

The manner of reporting shall always be factual. Numerical values and status to be presented must be taken purely from the final design review documents submitted and from the results of the independent review's analysis and assessments per applicable codes and standards.

The terms and phrases to be included in reporting any issue arising from the design review must be written carefully and reflect professionalism. The PR must not use offensive nor malicious words or phrases. Thus, the report must be factual and enlightening for the EOR and PR.

The assessment of each part of the report should avoid terms like erroneous, in error and misses, among others. Reporting should preferably be neutral, for example, statement for bars needing additional quantity may be stated "underestimated" and bars in element with quantities that maybe reduced may be stated that "bars are overestimated by as much as 25%".

The PR shall make comments that are clear, legible and complete so that the EOR will easily understand it. Clear comments will eliminate confusion and reduce time spent in back-check.

Mark-up and Comments

Generally, comments should be complete, clear and legible.

If possible, the PR should use words which would apply to numerous drawings so that the comments do not need to be repeated on each drawing.

When the PR makes the same specific comments at many different details, the comments should be identified by either creating a standard, numbered list of comments with the comment numbers referenced at each detail, or by marking the comments on each detail.

The PR may use 'paste-on' comments where applicable to save time and to maintain uniformity of comments.

Examples of Reviewer's Comments/Wordings

- Use specific comments such as: "*Show complete details in accordance with your calculation in pages 17 to 24.*"
- Do not use vague comments such as: "*Clarify welding.*"
- Avoid personalized wording such as: "*Your calculations for this connection is in error.*"
- Provide code references for comments whenever possible: "*Provide additional lath support at horizontal soffits per. . .*"
- If the properties of an element were improperly used in calculations and the element is overstressed, the PR should write a comment on the sheet where the overstressed element is shown such as: "*W18 x 36 overstressed. Recheck Section Modulus used in calculation. See AISC page.... and your calculation sheet F-19.*"
- The PR can make independent calculations when portions of the design professional's calculations are difficult to follow or interpret: "*Shear wall is overstressed along Gridline-A, wall shears is in excess to allowable by 13 kN/m.*"
- If the PR does extensive independent calculations, then he or she must number the calculations in sequence and mark the calculation page number on the comment to facilitate the back-check: "*Composite beam overstressed, recheck design loads. See page 28.*"

References

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APPENDIX 1-B

GUIDELINES AND IMPLEMENTING RULES ON EARTHQUAKE RECORDING INSTRUMENTATION FOR BUILDINGS

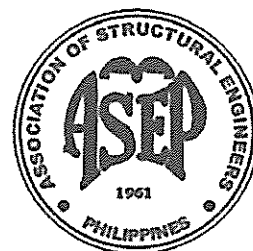
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APPENDIX I-B

GUIDELINES AND IMPLEMENTING RULES ON EARTHQUAKE RECORDING INSTRUMENTATION FOR BUILDINGS

I. INTRODUCTION

Technology on building instrumentation for seismic monitoring has improved tremendously in the past decade. The purpose of this Guidelines and Implementing Rules on Earthquake Recording Instrumentation for Buildings is to provide information on the specifications and uses of earthquake recording instruments or buildings as provided in Section 105.2 of the National Structural Code of the Philippines 2010 Volume 1, Sixth Edition [NSCP 2010].

The Guidelines and Implementing Rules on Earthquake Recording Instrumentation for Buildings provide earthquake instrumentation schemes for certain buildings to record building response during major seismic events for subsequent analysis. Adequate analysis of building response during earthquake is an important parameter in building safety evaluation in the confirmation and resumption of operations.

Installation of earthquake recording instruments first appeared in the National Structural Code of the Philippines 1992 Edition. At that period, structural engineers were mostly interested in the strength design capacity of the buildings based on seismic parameters provided in the Uniform Building Code (UBC) of the United States, a referral standard of the NSCP. This provision in the 1992 NSCP, however, was not enforced. Code developers started to recognize the importance of not only strength but serviceability in buildings as well. The experiences from the 1994 Northridge Earthquake in the US and the 1995 Kobe Earthquake in Japan gave credence to these considerations.

The NSCP 2010 states that *“Unless waived by the building official, every building in Seismic Zone 4 over fifty (50) meters in height shall be provided with not less than three (3) approved recording accelerograph. The accelerograph shall be interconnected for common start and common timing.”* Due to recent earthquakes and proliferation of high-rise buildings, the Philippines needs to have its own earthquake records for validating the seismic design parameters used, in order to support earthquake disaster mitigation / remedial efforts; thus, there is the need to implement the requirements for the earthquake recording instrumentation. Until such time that considerable sets of

adequate earthquake records have been obtained for various types of buildings or relevant provisions in the NSCP have been amended, the waiver stated above is temporarily suspended and buildings indicated in Table 1 shall be provided with earthquake recording instruments.

ASEP therefore deemed it necessary to improve our understanding of the building response based on real seismic event from local earthquake generators by promoting earthquake recording instrumentation for buildings as the NSCP provision was reiterated in the 2001 and 2010 Editions. Due to more recent developments in building instrumentation technology, a number of instrumentations are available to obtain the building response, and satisfy and comply with the objective of the NSCP Section 105 provisions. Hence, the requirement for three (3) accelerographs is further enhanced and modified to consider the latest and economical building instruments, thus, the combination or combinations of accelerographs, accelerometers, velocity meters and data loggers are considered. To measure building response due to long period earthquakes and distant sources normally critical to tall buildings, the addition of velocimeters is necessary.

To further address the disaster management effort in the country, essential facilities such as hospitals and some government buildings, which are important facilities in disaster response, are recommended to be instrumented. In addition, with this new provision, building response from low-rise structures can be obtained to determine building behavior due to near source or short period earthquakes.

III. OBJECTIVES OF THESE GUIDELINES

Section 102 of the National Building Code of the Philippines (PD 1096) states that *“It is hereby declared to be the policy of the State to safeguard life, health, property, and public welfare, consistent with the principles of sound environmental management and control; and to this end, make it the purpose of this Code to provide for all buildings and structures, a framework of minimum standards and requirements to regulate and control their location, site, design, quality of materials, construction, use, occupancy, and maintenance.”*

In conformance thereto and as provided in the NSCP 2010, these Guidelines and Implementing Rules on Earthquake Recording Instrumentation for Buildings is developed to improve the understanding of the actual dynamic behavior of buildings under earthquake loading and confirm the design according to the NSCP. The recorded data can be used to improve the structural code thereby reducing loss of lives and limbs as well as properties during future damaging earthquakes. The response data from several buildings in a particular area or several areas will also be used as basis for the government’s earthquake disaster mitigation/remedial and rehabilitation strategies including its emergency response and relief operations programs. The instruments may also be used to set off alarms at specified intensity levels. They may also be used to trigger automatic switching off utilities such as gas lines, electric power lines and elevators as may be prudent in case of high intensity earthquake. The recorded data are important parameters for building safety re-evaluation and resumption of operations including post-earthquake evaluation of buildings.

IV. DEFINITION OF TERMS

ACCELEROGRAPH are accelerograph records the acceleration of particles on the surface of the earth as a function of time, which is called an accelerogram. The accelerograph generally records three mutually perpendicular components of motion in the vertical and two orthogonal horizontal directions.

ACCELERATION is the rate at which the velocity of a particle changes with time.

ACCELEROMETER is an instrument used to measure acceleration in the vertical and two orthogonal horizontal directions. An accelerometer has no built-in data recording capacity and is attached to a multi-channel data logger or an accelerograph to record measured acceleration.

ACCREDITED STRUCTURAL ENGINEER (ASE) is a civil engineer with special qualifications to practice structural engineering with special training in earthquake engineering and certified by ASEP.

ACTIONS (GROUND MOTION) is a general term including all aspects of ground motion, namely acceleration, velocity, or displacement from an earthquake or other energy source.

BANDWIDTH is the frequency range that the sensor operates, measured in hertz. (Hz)

CHANNEL is a path along which information (as data or voice) in the form of electrical signal, passes; a band of frequencies of sufficient width for a single radio or television communication.

CLUSTERED BUILDINGS is a group of buildings built close together on a sizable tract of land in order to preserve open spaces larger than the individual yard for common recreation.

DAMPING is the energy dissipation properties of a material or system under cyclic stress.

DATA LOGGER is a data logger is an electronic device that records data over time or in relation to location either with a built in instrument or sensor or via external instruments and sensors.

DISPLACEMENT is the measured distance travelled by a particle from an initial position.

ENVIRONMENT is the aggregate of surrounding things, conditions, or influences that may affect the operability of an instrumentation device such as accelerograph, velocimeter, etc.

ERI. Earthquake Recording Instrumentations.

FFT. Fast Fourier Transform (FFT) is a numerical algorithm to compute the discrete Fourier transform (DFT) and its inverse. Fourier analysis converts time to frequency and vice versa; an FFT rapidly computes such transformations by factorizing the DFT matrix into a product of sparse factors.

GALS is the unit measure of acceleration equivalent to $(1/1000)*g$. Note that $1000 \text{ gals} = 1g$.

g is acceleration due to gravity equals to 9.81 m/s^2 or 32.2 ft/s^2 .

INTENSITY is a number (written as a Roman numeral) describing the severity of an earthquake in terms of its effects on the earth's surface and on humans and their structures.

INTENSITY METER is an intensity meter records and stores the various data that are associated with the earthquake and that it can notify those data to host system as it equips data communication function. In addition, it makes the "Earthquake Early Warning System" workable with creating a system network by making use of optional "earthquake early detecting function."

IP67. The Ingress Protection rating system is a classification system showing the degrees of protection of the instrumentation device from solid objects and liquids. The first number refers to the protection against solid objects, normally dust. If the first number is 0, there is no protection provided. A number 5 refers to limited protection against dust. The number 6 is for total protection against dust. The second number of the IP rating system refers to protection against liquids. A "0" indicates no protection, while a "7" refers to protection against immersion between 15 cm to 1 m for 30 minutes.

NATURAL FREQUENCY is the number of wave cycles per second which a system tends to oscillate in the absence of any driving or damping force.

PEAK GROUND ACCELERATION [PGA] is the maximum ground acceleration at a specific location for the time interval.

PERIOD is the time interval required for one full cycle of a wave.

REFUGE AREA is an area inside a building where people evacuate or assemble during a disaster or emergency i.e. fire, but not for earthquake.

RESPONSE SPECTRUM is a plot of the peak or amplitude of steady-state response (displacement, velocity or acceleration) of a series of oscillators of varying natural frequency that are forced into motion by the same base vibration or shock.

SIR. Seismic Instrumentation Room.

STRONG MOTION is a ground motion of sufficient amplitude to be of interest in evaluating the damage caused by earthquakes or nuclear explosions.

TIME HISTORY is the sequence of values of any time-varying quantity (such as a ground motion measurement) reckoned at a set of [usually] equal time intervals.

VELOCIMETER is an instrument used to measure velocity of a particle.

VELOCITY is a measure of the rate of motion of a particle expressed as the rate of change of its position in a particular direction with time.

V. EARTHQUAKE RECORDING INSTRUMENTATION REQUIREMENTS

The requirements of NSCP Section 105.2 shall apply to all existing buildings listed in Table 1, located in Seismic Zone 4 (entire Philippines except Palawan and Tawi-Tawi), for which certificates of occupancies were issued. Building permits shall only be issued for buildings qualified for seismic instrumentation when site or location of SIR has been indicated or incorporated in the plan.

TABLE 1. EARTHQUAKE RECORDING INSTRUMENTATION REQUIREMENTS		
TYPE AND HEIGHT OF BUILDING	LOCATION	REQUIREMENTS
GOVERNMENT BUILDINGS		
A. Hospitals, schools and other buildings fifty (50) meters high and above	- Three (3) accelerographs at Ground Floor / Lowest Basement; Middle Floor; and Floor Below Roof, or - One (1) accelerograph at Ground Floor / Lowest Basement interphased with two (2) accelerometers at Middle Floor and Floor Below Roof, or - Three (3) accelerometers with common data logger at Ground Floor / Lowest Basement; Middle Floor; and Floor Below Roof	- Accelerograph for recording waveform and transformed to FFT. - Data output to include acceleration response spectra and pseudo acceleration response. - With GPS capability. - Capability to send data to data center of the government.
B. Hospitals with 50-bed capacity or more and Schools with twenty (20) classrooms or more but not less than three (3) storey high	One (1) accelerograph or one (1) accelerometer connected to a data logger, at Ground Floor / Lowest Basement	
C. Provincial / City / Municipal Halls and Buildings	One (1) accelerograph or one (1) accelerometer connected to a data logger, at Ground Floor Level/Lowest Basement	

TABLE 1. EARTHQUAKE RECORDING INSTRUMENTATION REQUIREMENTS (continued)		
TYPE AND HEIGHT OF BUILDING	LOCATION	REQUIREMENTS
PRIVATE BUILDINGS		
A. Buildings fifty (50) meters high and above	1. Three (3) accelerographs at Ground Floor / Lowest Basement; Middle Floor; and Floor Below Roof, or 2. One (1) accelerograph at Ground Floor / Lowest Basement interphase with two (2) accelerometers at Middle Floor and Floor Below Roof, or 3. Three (3) accelerometers with common data logger at Ground Floor / Lowest Basement; Middle Floor; and Floor Below Roof	1. Accelerograph for recording waveform and transformed to FFT. 2. Data output to include acceleration response spectra and pseudo acceleration response. 3. For buildings above ninety (90) meters or thirty (30) storeys in height, additional velocity meter at ground floor / lowest basement shall be installed. Output data to include velocity response spectra and pseudo velocity response spectra. Data logger to be part of the system. 4. With GPS capability.
B. Hospitals with 50-bed capacity or more and Schools with 20 classrooms or more but not less than three (3) storey high	One (1) accelerograph or one (1) accelerometer connected to a data logger at Ground Floor / Lowest Basement	
C. Commercial Buildings with occupancy of at least one thousand (1,000) persons or gross floor area of at least ten thousand (10,000) square meters	One (1) accelerograph or one (1) accelerometer connected to a data logger at Ground Floor / Lowest Basement	

Table 1 shows the types of buildings required to be installed with earthquake recording instrumentation. The requirements for installation of accelerograph are for buildings located in cities and municipalities within a 200 km radius from a Type A faults as specified in the NSCP 2010 and as indicated from the active fault maps issued by the Philippine Institute of Seismology and Volcanology (PHIVOLCS).

For clustered buildings with completely similar design and construction, it shall have a minimum of eighteen (18) channels with a common data logger. The location of the instruments shall be determined by an Accredited Structural Engineer.

1. *Maintenance.* Earthquake Recording Instruments shall be maintained in proper working condition. The installation, servicing or removal of the instruments shall be done by qualified technical personnel of the supplier whose product complies with the minimum specifications as specified in these Guidelines and Implementing Rules.

Maintenance of the instruments shall be by the owner of the building subject to the monitoring of the Building Official or its designated representative.

2. *Service Period.* The maximum service interval is one (1) year. If the instrument is inoperative at two consecutive service inspections, then a re-inspection and servicing shall be required at a maximum service interval of six (6) months until the instrument is rendered fully operative. When the instrument continuously requires repair for a period of four (4) consecutive years, or inoperative repeatedly for at least three (3) times in a four (4)-year period, the instrument shall be replaced.
3. *Instrumentation of Selected Building.* All owners of existing buildings listed in Table 1 shall provide accessible space for the installation of appropriate earthquake recording instruments. Location of said instruments shall be determined by an Accredited Structural Engineer.

For proposed buildings, the Accredited Structural Engineer shall include the layout, instrument specifications, installation requirements, and location of the instrument in the structural plans submitted for building permit purposes.

The actual installation of the instruments shall be verified by the Building Official.

For existing buildings without ERI, the installation of these instruments shall form part of the requirements of the Annual Certificate of Inspection issued by the Building Official.

For existing buildings with ERI, the building owner shall be required to submit a certification from ASE that the existing ERI conform to these guidelines. If the existing ERI do not conform to these guidelines, the building owner shall upgrade such ERI.

For new buildings, the installation of these instruments shall form part of the requirements for Certificate of Occupancy issued by the Building Official.

4. *Additional Requisite Information of Buildings to be instrumented.* It is necessary to establish a baseline data to make effective use of the records to be collected from the accelerograph(s) installed in the building. The following information are required:
 - As-built blueprints,
 - Structural design calculations,
 - Dynamic analysis (mode shapes and frequencies) as used in the design calculations, if available, forced-vibration test results, and ambient-vibration test results, and
 - Comprehensive subsurface soil exploration and investigation report.

VI. DATA PROCESSING

Modern strong motion instruments have capabilities to store and transmit digital data through telecommunications links and other media, including the internet.

1. The data from digital recordings are passed through a correction algorithm that applies a high-frequency filter (50 Hz typical: 1 Hz=1 cycle per second). Plots of the corrected acceleration, velocity, and displacements for each channel of recording are prepared.
2. Response spectra are calculated for periods up to about half of the long-period limit. Linear plots of relative-acceleration response spectra and plots of pseudo-acceleration response are prepared if specified to the instrument supplier.
3. Fourier amplitude spectra, calculated by Fast Fourier Transform (FFT), are presented on linear axes and log-log axes. These sets of processed data are then provided to the user for evaluation, assessment of facilities and structures, and research.

VII. STANDARD SPECIFICATIONS / REQUIREMENTS FOR EARTHQUAKE RECORDING INSTRUMENTS

1. The following are the minimum specifications for Earthquake Recording Instruments to be used for buildings listed in Table 1.

A. Accelerographs/Accelerometers:

- Minimum design life: Ten (10) years and should be demonstrated and certified to have a 40,000-hour mean time (minimum) between failures.
- Minimum of three (3) components - Vertical, Longitudinal and Transverse
- Natural frequency: Above 50 Hz.
- Damping: Approximately 60 to 70 percent critical.
- Sensitivity: ± 2000 gals or $\pm 2g$ (full scale/V)
- Bandwidth: DC to 100 Hz
- Environment: IP67 or better
- Input Range: $\pm 2g$ - $\pm 6g$

B. Velocimeters

- Minimum of three (3) components - Vertical, Longitudinal and Transverse
- Natural frequency: Above 50 Hz.
- Damping: Approximately 60 to 70 percent critical.
- Sensitivity: ± 2 m/s
- Bandwidth: 0.1 Hz to 100 Hz
- Environment: IP67 or better

C. Data Logger / Recording. (Common for Accelerographs/Accelerometers and Velocimeters)

- Sampling Frequency: A minimum of 100 samples per second.
- Time: From at least 20 seconds before the ground shaking begins until 30 seconds after the last triggering level motion.
- RMS Noise: System noise shall be less than 40 μg 's measured over 0 – 30 Hz.
- Media: Digital storage media (minimum of 32 GB)

- Continuous Monitoring: Capable for continuous recording by minimum one (1) year
- AD converter: 24 bit or better.

D. Timing.

- Interval: Half second or less.
- Accuracy: Plus or minus 0.2 second per 100 seconds.
- Type: GPS or NTP server.

E. Triggering.

- Method: Pendulum or other device using earthquake motion as exciting force.
- Level: Accelerograph: 0.5 to 100 gals, nominal / Velocimeter: 0.005 mm/s to 1 mm/s
- Time: Full operation of accelerograph / velocimeter in not over 0.1 second after activation.

F. Power.

- Battery maintained by trickle charger from AC power and capable of powering the accelerograph and velocimeter for two (2) days after loss of power.

G. Communication

- Ethernet: 10 base-T or 100 base-TX
- Protocol: TCP/IP FTP/SFTP

2. Records. When media is used for recording, a new media load shall be placed in the instruments when the media remaining is less than 1/3 of original load. For instruments, memory should be copied out and emptied when the remaining amount is less than 1/3 of the original capacity.

3. Refurbishing and Replacement. When the instrument supplier finds that the instrument must be removed from the building for repair, the instrument shall be replaced by a temporary identical instrument, and the permanent instrument shall be returned and made operative within 60 days from the removal date.

4. Battery Inspection. The instrument shall be tested with any charge device disconnected from an electric power source.

VIII. LOCATION AND INSTALLATION OF THE INSTRUMENTS

1. General.

The instrument shall be located so that access by qualified technical personnel is maintained at all times and is unobstructed by room contents. A sign stating "MAINTAIN CLEAR ACCESS TO THIS INSTRUMENT" shall be posted in a conspicuous location. *No instruments shall be located in refuge area.*

The preferred locations of the instruments are in small, seldom-used rooms or closets near a column (in a vertically aligned stack), with adequate space to securely mount the instrument and an approved protective enclosure attached securely to the floor. The locations shall be marked on the submitted structural and architectural floor plans, and properly approved.

2. Buildings with three (3) or more instruments.

Buildings with three (3) or more accelerographs/accelerometers shall be located in the ground floor/lowest basement, middle floor, and the topmost floor of the building. When applicable, velocimeter shall be located in the lowest basement or ground floor level. The locations of the instruments are selected to provide the maximum information of the building response from a major earthquake. Such information would form part of the valuable data in understanding the building's behavior during major seismic event.

3. Orientation of the Instruments.

All instruments shall be installed with the same orientation relative to the building, with the orientation chosen such that the reference or long dimension of the instrument is aligned with a major axis of the building. The orientation of the instrument shall be clearly marked on the submitted structural plans. The supplier-installer shall certify that the instruments are oriented as per plan.

IX. DATA RETRIEVAL AND INTERPRETATION

Immediately after the occurrence of an Intensity VI earthquake or greater in the locality as determined by the Philippine Institute of Volcanology and Seismology (PHIVOLCS), the Building Official shall issue a written notice to the building owner to retrieve the data and to have the data interpreted by an Accredited Structural Engineer. The retrieval and interpretation of the data shall be performed by an Accredited Structural Engineer. The data and interpretation of the building shall be submitted by the Owner to the DPWH for storage, post-earthquake safety evaluation of the building, and for emergency response.

X. DATA STORAGE AND ARCHIVING

Data storage and archiving shall be at the DPWH Central Office or other data centers designated by the DPWH. The ASEP, upon written request to the DPWH, shall be provided the said data.

XI. CERTIFICATE OF INSTALLATION OF EARTHQUAKE RECORDING INSTRUMENTATION

Upon compliance of building owners of these Guidelines and Implementing Rules on Earthquake Recording Instrumentation, the Building Official shall issue a Certificate of Installation of Earthquake Recording Instrumentation. The Certificate must be posted at the room/s where the instrument is located and in a conspicuous place, properly protected/secured, in the ground floor lobby of the building.

XII. TESTING, INSPECTION AND COMMISSIONING

Building Owner, Building Official, and Supplier shall inspect, test, and commission the seismic monitoring system together to ensure that the systems are in proper operational condition and comply with the requirements of these guidelines. The Supplier must submit a certificate from the manufacturer that the instrument is in good working condition.

The Building Owner shall be responsible for the protection and maintenance of the site of the ERI as prescribed in these guidelines.

XIII. SUPPORT AND MAINTENANCE

The seismic monitoring system shall have a maintenance clearance as per the requirement of the National Structural Code of the Philippines under Section 105.2. "Maintenance and service shall be provided by the owner of the building."

"The supplier shall provide guarantee that the system shall have a maintenance period for at least 10 years. For the service period, the maximum service interval is one year. The equipment obsolescence shall not hinder the proper continuous operation of the equipment throughout the 10 years duration. When the equipment's supplier finds that the instrument must be removed from the building for repair, there must be a service unit as a temporary replacement to continue the collection of data, if and when there is an occurrence of an earthquake during the duration of the repair.

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Chapter 2

MINIMUM DESIGN LOADS

NATIONAL STRUCTURAL CODE OF THE PHILIPPINES VOLUME I BUILDINGS, TOWERS AND OTHER VERTICAL STRUCTURES

SEVENTH EDITION, 2015

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Table of Contents

SECTION 201	4
GENERAL REQUIREMENTS.....	4
201.1 Scope.....	4
SECTION 202	4
DEFINITIONS.....	4
SECTION 203	10
COMBINATIONS OF LOADS.....	10
203.1 General	10
203.2 Symbols and Notations.....	10
203.3 Load Combinations using Strength Design or Load and Resistance Factor Design	11
203.4 Load Combinations Using Allowable Stress or Allowable Strength Design	11
203.5 Special Seismic Load Combinations	11
SECTION 204	12
DEAD LOADS.....	12
204.1 General	12
204.2 Weights of Materials and Constructions	12
204.3 Partition Loads	12
SECTION 205	15
LIVE LOADS.....	15
205.1 General	15
205.2 Critical Distribution of Live Loads	15
205.3 Floor Live Loads	15
205.4 Roof Live Loads.....	19
205.5 Reduction of Live Loads	19
205.6 Alternate Floor Live Load Reduction.....	20
SECTION 206	21
OTHER MINIMUM LOADS.....	21
206.1 General	21
206.2 Other Loads	21
206.3 Impact Loads	21
206.4 Anchorage of Concrete and Masonry Walls.....	21
206.5 Interior Wall Loads	21
206.6 Retaining Walls.....	21
206.7 Water Accumulation	21
206.8 Uplift on Floors and Foundations.....	22
206.9 Crane Loads	22
206.10 Heliport and Helistop Landing Areas.....	22
SECTION 207	23
WIND LOADS.....	23
207.1 Specifications	23
207A General Requirements	23
207A.1 Procedures	23
207A.2 Definitions.....	25
207A.3 Symbols and Notations	29

207A.4 General	31
207A.5 Wind Hazard Map	32
207A.6 Wind Directionality	35
207A.7 Exposure	36
207A.8 Topographic Effects	46
207A.9 Gust Effects	49
207A.9.1 Gust Effect Factor	55
207A.10 Enclosure Classification	57
207A.11 Internal Pressure Coefficient	60
207B Wind Loads On Buildings—MWFRS (Directional Procedure)	61
207B.1 Scope	61
Part 1: Enclosed, Partially Enclosed, and Open Buildings of All Heights	62
207B.2 General Requirements	62
207B.3 Velocity Pressure	62
207B.4 Wind Loads—Main Wind Force-Resisting System	68
Part 2: Enclosed Simple Diaphragm Buildings with $h = 48$ m	82
207B.5 General Requirements	82
207B.6 Wind Loads—Main Wind Force-Resisting System	84
207C Wind Loads On Buildings—MWFRS (Envelope Procedure)	104
Part 1: Enclosed and Partially Enclosed Low-Rise Buildings	104
207C.2 General Requirements	104
207C.3 Velocity Pressure	105
207C.4 Wind Loads — Main Wind-Force Resisting System	109
Part 2: Enclosed Simple Diaphragm Low-Rise Buildings	112
207C.5 General Requirements	113
207C.6 Wind Loads - Main Wind-Force Resisting System	114
207D Wind Loads on Other Structures and Building Appurtenances - MWFRS	119
207D.2 General Requirements	119
207D.3 Velocity Pressure	119
207D.4 Design Wind Loads - Solid Freestanding Walls and Solid Signs	122
207D.5 Design Wind Loads—Other Structures	122
207D.6 Parapets	123
207D.8 Minimum Design Wind Loading	126
207E Wind Loads – Components and Cladding (C&C)	130
207E.1 Scope	130
207E.2 General Requirements	132
207E.3 Velocity Pressure	133
Part 1: Low-Rise Buildings	136
207E.4 Building Types	136
Part 2: Low-Rise Buildings (Simplified)	137
207E.5 Building Types	137
Part 3: Buildings with $h > 18$ m	139
207E.6 Building Types	139
Part 4: Buildings with $h \leq 48$ m (Simplified)	140
207E.7 Building Types	140
Part 5: Open Buildings	155
207E.8 Building Types	155
Part 6: Building Appurtenances and Rooftop Structures and Equipment	156
207E.9 Parapets	156
207E.10 Roof Overhangs	157
207E.11 Rooftop Structures and Equipment for Buildings with $h \leq 18$ m	158
207F Wind Tunnel Procedure	180
SECTION 208	184
EARTHQUAKE LOADS	184
208.1 General	184

208.2	Definitions.....	184
208.3	Symbols and Notations.....	184
208.4	Basis for Design	185
208.5	Minimum Design Lateral Forces and Related Effects.....	212
208.6	Earthquake Loads and Modeling Requirements.....	219
208.7	Detailed Systems Design Requirements.....	221
208.8	Non-Building Structures.....	229
208.9	Lateral Force on Elements of Structures, Nonstructural Components and Equipment Supported by Structures.....	231
208.10	Alternative Earthquake Load Procedure.....	236
SECTION 209		236
SOIL LATERAL LOADS.....		236
209.1	General	236
SECTION 210		238
RAIN LOADS		238
210.1	Roof Drainage	238
210.2	Design Rain Loads	238
210.3	Ponding Instability	238
210.4	Controlled Drainage	238
SECTION 211		238
FLOOD LOADS		238
211.1	General	238
211.2	Definitions.....	238
211.3	Design Requirements	239
211.4	Loads During Flooding	240
211.5	Establishment of Flood Hazard Areas	242
211.6	Design and Construction	242
211.7	Flood Hazard Documentation.....	242
211.8	Consensus Standards and Other Referenced Documents	243

SECTION 201 GENERAL REQUIREMENTS

201.1 Scope

This chapter provides minimum design load requirements for the design of buildings, towers and other vertical structures. Loads and appropriate load combinations which have been developed to be used together for strength design and allowable stress design are set forth.

SECTION 202 DEFINITIONS

The following terms are defined for use in this section:

ACCESS FLOOR SYSTEM is an assembly consisting of panels mounted on pedestals to provide an under-floor space for the installation of mechanical, electrical, communication or similar systems or to serve as an air-supply or return-air plenum.

AGRICULTURAL BUILDING is a structure designed and constructed to house farm implements, hay, grain, poultry, livestock or other horticultural products. The structure shall not be a place of human habitation or a place of employment where agricultural products are processed, treated, or packaged, nor shall it be a place used by the public.

ALLOWABLE STRESS DESIGN (ASD) is a method of proportioning and designing structural members such that elastically computed stresses produced in the members by nominal loads do not exceed specified allowable stresses (also called working stress design).

ASSEMBLY BUILDING is a building or portion of a building for the gathering together of 50 or more persons for such purposes as deliberation, education, instruction, worship, entertainment, amusement, drinking or dining, or awaiting transportation.

AWNING is an architectural projection that provides weather protection, identity, or decoration and is wholly supported by the building to which it is attached.

BALCONY, EXTERIOR, is an exterior floor system projecting from and supported by a structure without additional independent supports.

BASE is the level at which the earthquake motions are considered to be imparted to the structure or the level at which the structure, as a dynamic vibrator, is supported.

BASE SHEAR is the total design lateral force or shear at the base of a structure.

BASIC WIND SPEED is a three-second gust speed at 10 m above the ground in Exposure C (see Section 207A.7.3) as determined in accordance with Section 207A.5.1 and associated with an annual probability of 0.02, (i.e. 50-year mean recurrence interval).

BEARING WALL SYSTEM is a structural system that does not have a complete vertical load-carrying space frame. See Section 208.4.6.1.

BOUNDARY ELEMENT is an element at edges of openings or at perimeters of shear walls or diaphragms.

BRACED FRAME is essentially a vertical truss system of the concentric or eccentric type that is provided to resist lateral forces.

BUILDING FRAME SYSTEM is essentially a complete space frame that provides support for gravity loads. See Section 208.4.6.2.

BRACED WALL LINE is a series of braced wall panels in a single storey that meets the requirements of Section 620.10.3.

BRACED WALL PANEL is a section of wall braced in accordance with Section 620.10.3.

BUILDING, ENCLOSED is a building that does not comply with the requirements for open or partially enclosed buildings.

BUILDING ENVELOPE refers to cladding, roofing, exterior wall, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.

BUILDING, FLEXIBLE refers to slender buildings that have a fundamental natural frequency less than 1.0 Hz.

BUILDING, LOW-RISE is an enclosed or partially enclosed building that complies with the following conditions:

1. Mean roof height, h , less than or equal to 18m, and
2. Mean roof height, h , does not exceed least horizontal dimension.

BUILDING, OPEN refers to a building having each wall at least 80 percent open. This condition is expressed for each wall by the equation $A_o \geq 0.8A_g$. See symbols and notations.

BUILDING, PARTIALLY ENCLOSED is a building that complies with both of the following conditions:

1. the total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10%; and
2. The total area of openings in a wall that receives positive external pressure exceeds 0.5 m^2 or 1 percent of the area of that wall, whichever is smaller, and the

percentage of openings in the balance of the building envelope does not exceed 20 percent.

These conditions are expressed by the following equations:

1. $A_o > 1.10A_{oi}$
2. $A_o > \text{smaller of } (0.5 \text{ m}^2 \text{ or } 0.01A_g)$
3. $A_{oi}/A_{gi} \leq 0.20$

See symbols and notations.

BUILDING OR OTHER STRUCTURE, REGULAR-SHAPED refers to a building or other structure having no unusual geometrical irregularity in spatial form.

BUILDING OR OTHER STRUCTURES, RIGID refer to a building or other structure whose fundamental frequency is greater than or equal to 1.0 Hz.

BUILDING, SIMPLE DIAPHRAGM refers to a building in which both windward and leeward wind loads are transmitted through floor and roof diaphragms to the same vertical MWFRS (e.g., no structural separations).

CANTILEVERED COLUMN ELEMENT is a column element in a lateral-force-resisting system that cantilevers from a fixed base and has minimal moment capacity at the top, with lateral forces applied essentially at the top.

COLLECTOR is a member or element provided to transfer lateral forces from a portion of a structure to vertical elements of the lateral-force-resisting system.

COMPONENT is a part or element of an architectural, electrical, mechanical or structural system.

COMPONENT, EQUIPMENT is a mechanical or electrical component or element that is part of a mechanical and/or electrical system.

COMPONENT, FLEXIBLE is a component, including its attachments, having a fundamental period greater than 0.06 s.

COMPONENT, RIGID is a component, including its attachments, having a fundamental period less than or equal to 0.06s.

COMPONENTS AND CLADDING refers to elements of the building envelope that do not qualify as part of the MWFRS.

CONCENTRICALLY-BRACED FRAME is a braced frame in which the members are subjected primarily to axial forces.

CONVENTIONAL LIGHT-FRAME CONSTRUCTION is a type of construction in which the primary structural elements are formed by a system of repetitive wood framing members.

COVERING, IMPACT-RESISTANT is a covering designed to protect impact-resistant glazing.

CRIPPLE WALL is a framed stud wall extending from the top of the foundation to the underside of floor framing for the lowest occupied level.

DEAD LOADS consist of the weight of all materials and fixed equipment incorporated into the building or other structure.

DECK is an exterior floor system supported on at least two opposing sides by an adjacent structure and/or posts, piers, or other independent supports.

DESIGN BASIS GROUND MOTION is that ground motion that has a 10 percent chance of being exceeded in 50 years as determined by a site-specific hazard analysis or may be determined from a hazard map.

DESIGN FORCE is the equivalent static force to be used in the determination of wind loads for open buildings and other structures.

DESIGN RESPONSE SPECTRUM is an elastic response spectrum for 5 percent equivalent viscous damping used to represent the dynamic effects of the Design Basis Ground Motion for the design of structures in accordance with Sections 208.5 and 208.5.3.

DESIGN SEISMIC FORCE is the minimum total strength design base shear, factored and distributed in accordance with Section 208.5.

DESIGN PRESSURE is the equivalent static pressure to be used in the determination of wind loads for buildings.

DIAPHRAGM is a horizontal or nearly horizontal system acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes horizontal bracing systems.

DIAPHRAGM, BLOCKED is a diaphragm in which all sheathing edges not occurring on framing members are supported on and connected to blocking.

DIAPHRAGM CHORD or SHEAR WALL CHORD is the boundary element of a diaphragm or shear wall that is assumed to take axial stresses analogous to the flanges of a beam.

DIAPHRAGM STRUT (drag strut, tie, and collector) is the element of a diaphragm parallel to the applied load that collects and transfers diaphragm shear to the vertical resisting elements or distributes loads within the diaphragm. Such members may take axial tension or compression.

DIAPHRAGM, UNBLOCKED is a diaphragm that has edge nailing at supporting members only. Blocking between supporting structural members at panel edges is not included.

DRIFT or STOREY DRIFT is the lateral displacement of one level relative to the level above or below.

DUAL SYSTEM is a combination of moment-resisting frames and shear walls or braced frames designed in accordance with the criteria of Section 208.4.6.4.

EAVE HEIGHT is the distance from the ground surface adjacent to the building to the roof eave line at a particular wall. If the height of the eave varies along the wall, the average height shall be used.

ECCENTRICALLY BRACED FRAME (EBF) is a steel-braced frame designed in conformance with Section 528.

EFFECTIVE WIND AREA is the area used to determine GC_p . For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

ELASTIC RESPONSE PARAMETERS are forces and deformations determined from an elastic dynamic analysis using an unreduced ground motion representation, in accordance with Section 208.5.3.

ESCARPMENT, also known as scarp, with respect to topographic effect in Section 207A.8, is a cliff or steep slope generally separating two levels or gently sloping areas (see Figure 207A-8-1).

ESSENTIAL FACILITIES are buildings, towers and other vertical structures that are intended to remain operational in the event of extreme environmental loading from wind or earthquakes.

FACTORED LOAD is the product of a load specified in Sections 204 through 208 and a load factor. See Section 203.3 for combinations of factored loads.

FIBERBOARD is a fibrous, homogeneous panel made from lignocellulosic fibers (usually wood or sugar cane bagasse) and having a density of less than 50 kg/m^3 but more than 160 kg/m^3 .

FLEXIBLE ELEMENT or **SYSTEM** is one whose deformation under lateral load is significantly larger than adjoining parts of the system. Limiting ratios for defining specific flexible elements are set forth in Section 208.5.1.3.

FOREST PRODUCTS RESEARCH AND DEVELOPMENT INSTITUTE (FPRDI) is the Department of Science and Technology's (DOST's) research and development arm on forest products utilization. It is mandated to conduct basic and applied research to help the wood-using industries disseminate information and technologies on forest products to end users.

FREE ROOF is a roof with a configuration generally conforming to those shown in Figures 207B.4-4 through 207B.4-6 (monoslope, pitched, or troughed) in an open building with no enclosing walls underneath the roof surface.

GARAGE is a building or portion thereof in which motor vehicle containing flammable or combustible liquids or gas in its tank is stored, repaired or kept.

GARAGE, PRIVATE, is a building or a portion of a building, not more than 90 m^2 in area, in which only motor vehicles used by the tenants of the building or buildings on the premises are kept or stored.

GLAZING is a glass or transparent or translucent plastic sheet used in windows, doors, skylights, or curtain walls.

GLAZING, IMPACT-RESISTANT is a glazing that has been tested in accordance with ASTM E1886 and ASTM E1996 or other approved test methods to withstand the impact of wind-borne missiles likely to be generated in wind-borne debris regions during design winds.

GLUED BUILT-UP MEMBERS are structural elements, the section of which is composed of built-up lumber, wood structural panels or wood structural panels in combination with lumber, all parts bonded together with structural adhesives.

GRADE (LUMBER) is the classification of lumber in regard to strength and utility in accordance with the grading rules of an approved lumber grading agency.

HARDBOARD is a fibrous-felted, homogeneous panel made from lignocellulosic fibers consolidated under heat and pressure in a hot press to a density not less than 50 kg/m^3 .

HILL, with respect to topographic effects in Section 207A.8, is a land surface characterized by strong relief in any horizontal direction (Figure 207A.8-2).

HORIZONTAL BRACING SYSTEM is a horizontal truss system that serves the same function as a diaphragm.

IMPACT-RESISTANT COVERING, is a covering designed to protect glazing, which has been shown by testing in accordance with ASTM E1886 and ASTM E1996 of other approved test methods to withstand the impact or wind-borne debris missiles likely to be generated in wind-borne debris regions during design winds.

IMPORTANCE FACTOR is a factor that accounts for the degree of hazard to human life and damage to property.

INTERMEDIATE MOMENT RESISTING FRAME (IMRF) is a concrete frame designed in accordance with Section 412.

LATERAL-FORCE-RESISTING SYSTEM is that part of the structural system designed to resist the Design Seismic Forces.

LIMIT STATE is a condition beyond which a structure or member becomes unfit for service and is judged to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

LIVE LOADS are those loads produced by the use and occupancy of the building or other structure and do not include dead load, construction load, or environmental loads.

LOADS are forces or other actions that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movements, and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude. All other loads are variable loads.

LOAD AND RESISTANCE FACTOR DESIGN (LRFD) METHOD is a method of proportioning and designing structural elements using load and resistance factors such that no applicable limit state is reached when the structure is subjected to all appropriate load combinations. The term "LRFD" is used in the design of steel structures.

MACHINE-GRADED LUMBER (MGL) is a lumber evaluated by a machine using a non-destructive test and sorted into different stress grades.

MAIN WIND-FORCE RESISTING SYSTEM (MWFRS) is an assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.

MARQUEE is a permanent roofed structure attached to and supported by the building and projecting over public right-of-way.

MEAN ROOF HEIGHT is the average of the roof eave height and the height to the highest point on the roof surface, except that, for roof angles of less than or equal to 10°, the mean roof height shall be the roof eave height.

MOISTURE CONTENT (MC) is the amount of moisture in wood, usually measured as the percentage of water to the oven dry weight of the wood.

MOMENT-RESISTING FRAME is a frame in which members and joints are capable of resisting forces primarily by flexure.

MOMENT-RESISTING WALL FRAME (MRWF) is a masonry wall frame especially detailed to provide ductile behavior and designed in conformance with Section 708.2.6.

NOMINAL LOADING is a design load that stressed a member or fastening to the full allowable stress tabulated in this chapter. This loading may be applied for approximately 10 years, either continuously or cumulatively, and 90 percent of this load may be applied for the remainder of the life of the member or fastening.

NOMINAL SIZE (Lumber) refers to the commercial size designation of width and depth, in standard sawn lumber and glued laminated lumber grades; somewhat larger than the standard net size of dressed lumber.

OPENINGS are apertures or holes in the building envelope that allow air to flow through the building envelope and that are designed as “open” during design winds as defined by these provisions.

ORDINARY BRACED FRAME (OBF) is a steel-braced frame designed in accordance with the provisions of Section 527 or 528 or concrete-braced frame designed in accordance with Section 421.

ORDINARY MOMENT-RESISTING FRAME (OMRF) is a moment-resisting frame not meeting special detailing requirements for ductile behavior.

ORTHOGONAL EFFECTS are the earthquake load effects on structural elements simultaneously occurring to the lateral-force-resisting systems along two orthogonal axes.

OVERSTRENGTH is a characteristic of structures where the actual strength is larger than the design strength. The degree of over-strength is material-and system-dependent.

PARTICLEBOARD is a manufactured panel product consisting of particles of wood or combinations of wood particles and wood fibers bonded together with synthetic resins or other suitable bonding system by a bonding process, in accordance with approved nationally recognized standard.

PLYWOOD is a panel of laminated veneers conforming to Philippine National Standards (PNS 196) “Plywood Specifications”.

$P\Delta$ EFFECT is the secondary effect on shears, axial forces and moments of frame members induced by the horizontal displacement of vertical loads from various loading, when a structure is subjected to lateral forces.

RECOGNIZED LITERATURE are published research findings and technical papers that are approved.

RIDGE, with respect to topographic effects in Section 207A.8, is an elongated crest of a hill characterized by strong relief in two directions (see Figure 207A.8-1).

ROTATION is the torsional movement of a diaphragm about a vertical axis.

SHEAR WALL is a wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as vertical diaphragm or structural wall).

SHEAR WALL-FRAME INTERACTIVE SYSTEM uses combinations of shear walls and frames designed to resist lateral forces in proportion to their relative rigidities, considering interaction between shear walls and frames on all levels.

SHEATHING is a layer of boards or of other wood or fiber materials applied to the outer studs, joists, and rafters of a building to strengthen structures and serve as a base for an exterior weatherproof cladding.

SHEATHING, WALL is a layer of boards or of other wood or fiber materials used to cover the wall studding.

STRUCTURAL GLUED-LAMINATED TIMBER is any member comprising an assembly of laminations of lumber in which the grain of all laminations is approximately parallel longitudinally, in which the laminations are bonded with adhesives.

SUBDIAPHRAGM is a portion of a diaphragm used to transfer wall anchorage forces to diaphragm cross ties. It also refers to a portion of a larger wood diaphragm designed to anchor and transfer local forces to primary diaphragm struts and the main diaphragm.

SOFT STOREY is one in which the lateral stiffness is less than 70 percent of the stiffness of the storey above. See Table 208-9.

SPACE FRAME is a three-dimensional structural system, without bearing walls, composed of members interconnected so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor-bracing systems.

SPECIAL CONCENTRICALLY BRACED FRAME (SCBF) is a steel-braced frame designed in conformance with the provisions of Section 526.

SPECIAL MOMENT-RESISTING FRAME (SMRF) is a moment-resisting frame specially detailed to provide ductile behavior and comply with the requirements given in Chapter 4 or 5.

SPECIAL TRUSS-MOMENT FRAME (STMF) is a moment-resisting frame specially detailed to provide ductile behavior and comply with the provisions of Section 525.

STOREY is the space between levels. Storey x is the storey below level x .

STOREY DRIFT RATIO is the storey drift divided by the storey height.

STOREY SHEAR, V_x , is the summation of design lateral forces above the storey under consideration.

STRENGTH is the capacity of an element or a member to resist factored load as specified in Chapters 2, 3, 4, 5 and 7.

STRUCTURE is an assemblage of framing members designed to support gravity loads and resist lateral forces. Structures may be categorized as building structures or nonbuilding structures.

STRENGTH DESIGN is a method of proportioning and designing structural members such that the computed forces produced in the members by the factored load do not exceed the member design strength. The term strength design is used in the design of concrete structures.

TOWERS AND OTHER STRUCTURES are nonbuilding structures including poles, masts and billboards that are not typically occupied by persons but are also covered by this code.

TREATED WOOD is wood treated with an approved preservative under treating and quality control procedures.

VERTICAL LOAD-CARRYING FRAME is a space frame designed to carry vertical gravity loads.

WALL ANCHORAGE SYSTEM is the system of elements anchoring the wall to the diaphragm and those elements within the diaphragm required to develop the anchorage forces, including sub-diaphragms and continuous ties, as specified in Sections 208.7.2.7 and 208.7.2.8.

WALL, BEARING is any wall meeting either of the following classifications:

1. Any metal or wood stud wall that supports more than 1.45 kN/m of vertical load in addition to its own weight.
2. Any masonry or concrete wall that supports more than 2.90 kN/m of vertical load in addition to its own weight.

WALL, EXTERIOR is any wall or element of a wall, or any member or group of members, that defines the exterior boundaries or courts of a building and that has a slope of 60 degrees or greater with the horizontal plane.

WALL, NONBEARING is any wall that is not a bearing wall.

WALL, PARAPET is that part of any wall entirely above the roof line.

WALL, RETAINING is a wall designed to resist the lateral displacement of soil or other materials.

WEAK STOREY is one in which the storey strength is less than 80 percent of the storey above. See Table 208-9.

WIND-BORNE DEBRIS REGIONS are areas within typhoon prone regions located at:

1. Within 1.6 km of the coastal mean high water line where the basic wind speed is equal to or greater than 200 kph, or
2. In areas where the basic wind speed is equal to or greater than 250 kph.

WOOD OF NATURAL RESISTANCE TO DECAY OR TERMITES is the heartwood of the species set forth below. Corner sapwood is permitted on 5 percent of the pieces provided 90 percent or more of the width of each side on which it occurs is heartwood. Recognized species are:

- Decay resistant: Narra, Kamagong, Dao, Tangile.
- Termite resistant: Narra, Kamagong.

WOOD STRUCTURAL PANEL is a structural panel product composed primarily of wood and meeting the UBC Standard 23-2 and 23-3 or equivalent requirements of Philippine National Standards (PNS). Wood structural panels include all-veneer plywood, composite panels containing a combination of veneer and wood-based material, and mat-formed panel such as oriented stranded board and wafer board.

WYTHER is the portion of a wall which is one masonry unit in thickness. A collar joint is not considered a wythe.

SECTION 203 COMBINATIONS OF LOADS

203.1 General

Buildings, towers and other vertical structures and all portions thereof shall be designed to resist the load combinations specified in Section 203.3, 203.4 and 203.5.

The most critical effect can occur when one or more of the contributing loads are not acting. All applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations.

203.2 Symbols and Notations

<i>D</i>	=	dead load
<i>E</i>	=	earthquake load set forth in Section 208.6.1
<i>E_m</i>	=	estimated maximum earthquake force that can be developed in the structure as set forth in Section 208.6.1
<i>F</i>	=	load due to fluids with well-defined pressures and maximum heights
<i>H</i>	=	load due to lateral pressure of soil and water in soil
<i>L</i>	=	live load, except roof live load, including any permitted live load reduction
<i>L_r</i>	=	roof live load, including any permitted live load reduction
<i>P</i>	=	ponding load
<i>R</i>	=	rain load on the undeflected roof
<i>T</i>	=	self-straining force and effects arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement, or combinations thereof
<i>W</i>	=	load due to wind pressure

203.3 Load Combinations using Strength Design or Load and Resistance Factor Design

203.3.1 Basic Load Combinations

Where strength design or load and resistance factor design is used, structures and all portions thereof shall resist the most critical effects from the following combinations of factored loads:

$$1.4(D + F) \quad (203-1)$$

$$1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } R) \quad (203-2)$$

$$1.2D + 1.6(L_r \text{ or } R) + (f_1 L \text{ or } 0.5W) \quad (203-3)$$

$$1.2D + 1.0W + f_1 L + 0.5(L_r \text{ or } R) \quad (203-4)$$

$$1.2D + 1.0E + f_1 L \quad (203-5)$$

$$0.9D + 1.0W + 1.6H \quad (203-6)$$

$$0.9D + 1.0E + 1.6H \quad (203-7)$$

where

$$\begin{aligned} f_1 &= 1.0 \text{ for floors in places of public assembly,} \\ &\quad \text{for live loads in excess of 4.8 kPa, and for} \\ &\quad \text{garage live load, or} \\ &= 0.5 \text{ for other live loads} \end{aligned}$$

203.3.2 Other Loads

Where P is to be considered in design, the applicable load shall be added to Section 203.3.1 factored as $1.2P$.

203.4 Load Combinations Using Allowable Stress or Allowable Strength Design

203.4.1 Basic Load Combinations

Where allowable stress or allowable strength design is used, structures and all portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$$D + F \quad (203-8)$$

$$D + H + F + L + T \quad (203-9)$$

$$D + H + F + (L_r \text{ or } R) \quad (203-10)$$

$$D + H + F + 0.75[L + T(L_r \text{ or } R)] \quad (203-11)$$

$$D + H + F + \left(0.6W \text{ or } \frac{E}{1.4}\right) \quad (203-12)$$

No increase in allowable stresses shall be used with these load combinations except as specifically permitted by Section 203.4.2.

203.4.2 Alternate Basic Load Combinations

In lieu of the basic load combinations specified in Section 203.4.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following load combinations. When using these alternate basic load combinations, a one-third increase shall be permitted in allowable stresses for all combinations, including W or E .

$$D + H + F + 0.75 \left[L + L_r \left(0.6W \text{ or } \frac{E}{1.4} \right) \right] \quad (203-13)$$

$$0.6D + 0.6W + H \quad (203-14)$$

$$0.6D + \frac{E}{1.4} + H \quad (203-15)$$

$$D + L + (L_r \text{ or } R) \quad (203-16)$$

$$D + L + 0.6W \quad (203-17)$$

$$D + L + \frac{E}{1.4} \quad (203-18)$$

Exception:

Crane hook loads need not be combined with roof live load or with more than one-half of the wind load.

203.4.3 Other Loads

Where P is to be considered in design, each applicable load shall be added to the combinations specified in Sections 203.4.1 and 203.4.2.

203.5 Special Seismic Load Combinations

For both allowable stress design and strength design for concrete, and Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD) for steel, the following special load combinations for seismic design shall be used as specifically required by Section 208, or by Chapters 3 through 7.

$$1.2D + f_1 L + 1.0E_m \quad (203-19)$$

$$0.9D \pm 1.0E_m \quad (203-20)$$

where

- f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 4.8 kPa, and for garage live load, or
 = 0.5 for other live loads
 E_m = the maximum effect of horizontal and vertical forces as set forth in Section 208.6.1

SECTION 204 DEAD LOADS

204.1 General

Dead loads consist of the weight of all materials of construction incorporated into the building or other structure, including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items, and fixed service equipment, including the weight of cranes.

204.2 Weights of Materials and Constructions

The actual weights of materials and constructions shall be used in determining dead loads for purposes of design. In the absence of definite information, it shall be permitted to use the minimum values in Tables 204-1 and 204-2.

204.3 Partition Loads

Floors in office buildings and other buildings where partition locations are subject to change shall be designed to support, in addition to all other loads, a uniformly distributed dead load equal to 1.0 kPa.

Exception:

Access floor systems shall be designed to support, in addition to all other loads, a uniformly distributed dead load not less than 0.5 kPa.

Table 204-1 Minimum Densities for Design Loads from Materials (kN/m³)

Material	Density	Material	Density
Aluminum	26.7	Lime	
Bituminous products		Hydrated, loose	5.0
Asphaltum	12.7	Hydrated, compacted	7.1
Graphite	21.2	Masonry, Ashlar Stone	
Paraffin	8.8	Granite	25.9
Petroleum, crude	8.6	Limestone, crystalline	25.9
Petroleum, refined	7.9	Limestone, oolitic	21.2
Petroleum, benzine	7.2	Marble	27.2
Petroleum, gasoline	6.6	Sandstone	22.6
Pitch	10.8	Masonry, Brick	
Tar	11.8	Hard, low absorption	20.4
Brass	82.6	Medium, medium absorption	18.1
Bronze	86.7	Soft, high absorption	15.7
Cast-stone masonry (cement, stone, sand)	22.6	Masonry, Concrete (solid portion)	
Cement, portland, loose	14.1	Lightweight units	16.5
Ceramic tile	23.6	Medium weight units	19.6
Charcoal	1.9	Normal weight units	21.2
Cinder fill	9.0	Masonry grout	22.0
Cinders, dry, in bulk	7.1	Masonry, Rubble Stone	
Coal		Granite	24.0
Anthracite, piled	8.2	Limestone, crystalline	23.1
Bituminous, piled	7.4	Limestone, oolitic	21.7
Lignite, piled	7.4	Marble	24.5
Peat, dry, piled	3.6	Sandstone	21.5
Concrete, plain		Mortar, cement or lime	20.4
Cinder	17.0	Particle board	7.1
Expanded-slag aggregate	15.7	Plywood	5.7
Haydite, burned-clay aggregate	14.1	Riprap, not submerged	
Slag	20.7	Limestone	13.0
Stone	22.6	Sandstone	14.1
Vermiculite and perlite aggregate, nonload-bearing	3.9-7.9	Sand	
Other light aggregate, load bearing	11.0-16.5	Clean and dry	14.1
Concrete, reinforced		River, dry	16.7
Cinder	17.4	Slag	
Slag	21.7	Bank	11.0
Stone, including gravel	23.6	Bank screenings	17.0
Copper	87.3	Machine	15.1
Cork, compressed	2.2	Sand	8.2
Earth, not submerged		Slate	27.0
Clay, dry	9.9	Steel, cold-drawn	77.3
Clay, damp	17.3	Stone, quarried, piled	
Clay and gravel, dry	15.7	Basalt, granite, gneiss	15.1
Silt, moist, loose	12.3	Limestone, marble, quartz	14.9
Silt, moist, packed	15.1	Sandstone	12.9
Silt, flowing	17.0	Shale	14.5
Sand and gravel, dry, loose	15.7	Greenstone, hornblende	16.8
Sand and gravel, dry, packed	17.3	Terracotta, architectural	
Sand and gravel, wet	18.9	Voids filled	18.9
Earth, submerged		Voids unfilled	11.3
Clay	12.6	Tin	72.1
Soil	11.0	Water	
River mud	14.1	Fresh	9.8
Sand or gravel	9.4	Sea	10.1
Sand or gravel and clay	10.2	Wood (see Chapter 6 for relative densities for Philippine wood)	
Glass	25.1	Zinc, rolled sheet	70.5
Gravel, dry	16.3		
Gypsum, loose	11.0		
Gypsum, wallboard	7.9		
Ice	9.0		
Iron			
Cast	70.7		
Wrought	75.4		
Lead	111.5		

Table 204-2 Minimum Design Dead Loads (kPa)

Table 204-2 Minimum Design Dead Loads (kN/m²)						
Component		Load	Component		Load	
CEILINGS			FLOOR FILL			
Acoustical fiber board		0.05	Cinder concrete, per mm		0.017	
Gypsum board (per mm thickness)		0.008	Lightweight concrete, per mm		0.015	
Mechanical duct allowance		0.20	Sand, per mm		0.015	
Plaster on tile or concrete		0.24	Stone concrete, per mm		0.023	
Plaster on wood lath		0.38	FLOOR AND FLOOR FINISHES			
Suspended steel channel system		0.10	Asphalt block (50 mm), 13 mm mortar		1.44	
Suspended metal lath and cement plaster		0.72	Cement finish (25 mm) on stone-concrete fill		1.53	
Suspended metal lath and gypsum plaster		0.48	Ceramic or quarry tile (20 mm) on 13 mm mortar bed		0.77	
Wood furring suspension system		0.12	Ceramic or quarry tile (20 mm) on 25 mm mortar bed		1.10	
COVERINGS, Roof and Wall			Concrete fill finish (per mm thickness)		0.023	
Asphalt shingles		0.10	Hardwood flooring, 22 mm		0.19	
Cement tile		0.77	Linoleum or asphalt tile, 6mm		0.05	
Clay tile (for mortar add 0.48 kPa)			Marble and mortar on stone-concrete fill		1.58	
Book tile, 50 mm		0.57	Slate (per mm thickness)		0.028	
Book tile, 75 mm		0.96	Solid flat tile on 25-mm mortar base		1.10	
Ludowici		0.48	Subflooring, 19 mm		0.14	
Roman		0.57	Terrazzo (38 mm) directly on slab		0.91	
Spanish		0.91	Terrazzos (25 mm) on stone-concrete fill		1.53	
Composition:			Terrazzo (25 mm) on 50-mm stone concrete		1.53	
Three-ply ready roofing		0.05	Wood block (75 mm) on mastic, no fill		0.48	
Four-ply felt and gravel		0.26	Wood block (75 mm) on 13-mm mortar base		0.77	
Five-ply felt and gravel		0.29	FRAME PARTITIONS			
Copper or tin		0.05	Movable partitions		0.24	
Corrugated asbestos-cement roofing		0.19	Movable partitions (steel)		0.19	
Deck, metal 20 gage		0.12	Wood or steel studs, 13 mm gypsum board each side		0.38	
Deck, metal, 18 gage		0.14	Wood studs, 50 x 100, unplastered		0.19	
Fiberboard, 13mm		0.04	Wood studs 50 x 100, plastered one side		0.57	
Gypsum sheathing, 13 mm		0.10	Wood studs 50 x 100, plastered two side		0.96	
Insulation, roof boards (per mm thickness)			FRAME WALLS			
Cellular glass		0.0013	Exterior stud walls:			
Fibrous glass		0.0021	50x100 @ 400mm, 15 mm gypsum, insulated, 10 mm siding		0.53	
Fiberboard		0.0028	50x150 @ 400mm, 15 mm gypsum, insulated, 10 mm siding		0.57	
Perlite		0.0015	Exterior stud wall with brick veneer		2.30	
Polystyrene foam		0.0004	Windows, glass, frame and sash		0.38	
Urethane foam with skin	...	0.0009	Clay brick wythes:			
Plywood (per mm thickness)	0.0060		100 mm		1.87	
Rigid insulation, 13 mm		0.04	200 mm		3.74	
Skylight, metal frame,			300 mm		5.51	
10mm wire glass		0.38	400 mm		7.48	
Slate, 5 mm		0.34	CONCRETE MASONRY UNITS			
Slate, 6 mm		0.48	Hollow Concrete Masonry Units Unplastered. Add 0.24 kPa for each face plastered			
Waterproofing membranes:			Grout Spacing			
Bituminous, gravel-covered	0.26		Wythe thickness (mm)			
Bituminous, smooth surface	0.07		100 150 200			
Liquid, applied	0.05		16.5-kN/m³ Density of Unit			
Single-ply, sheet	0.03		No grout	1.05	1.15	1.48
Wood sheathing (per mm thickness)	0.0057		800	1.40	1.53	2.01
Wood shingles	0.14		600	1.50	1.63	2.20
			400	1.79	1.92	2.54
			Full	2.50	2.63	3.59
			19.6-kN/m³ Density of Unit			
			No grout	1.24	1.34	1.72
			800	1.59	1.72	2.25
			600	1.69	1.87	2.44
			400	1.98	2.11	2.82
			Full	2.69	2.82	3.88
			21.2-kN/m³ Density of Unit			
			No grout	1.39	1.44	1.87
			800	1.74	1.82	2.39
			600	1.83	1.96	2.59
			400	2.13	2.2	2.92
			Full	2.84	2.97	3.97

**SECTION 205
LIVE LOADS****205.1 General**

Live loads shall be the maximum loads expected by the intended use or occupancy but in no case shall be less than the loads required by this section.

205.2 Critical Distribution of Live Loads

Where structural members are arranged to create continuity, members shall be designed using the loading conditions, which would cause maximum shear and bending moments. This requirement may be satisfied in accordance with the provisions of Section 205.3.2 or 205.4.2, where applicable.

205.3 Floor Live Loads**205.3.1 General**

Floors shall be designed for the unit live loads as set forth in Table 205-1. These loads shall be taken as the minimum live loads of horizontal projection to be used in the design of buildings for the occupancies listed, and loads at least equal shall be assumed for uses not listed in this section but that creates or accommodates similar loadings.

Where it can be determined in designing floors that the actual live load will be greater than the value shown in Table 205-1, the actual live load shall be used in the design of such buildings or portions thereof. Special provisions shall be made for machine and apparatus loads.

205.3.2 Distribution of Uniform Floor Loads

Where uniform floor loads are involved, consideration may be limited to full dead load on all spans in combination with full live load on adjacent spans and alternate spans.

205.3.3 Concentrated Loads

Floors shall be designed to support safely the uniformly distributed live loads prescribed in this section or the concentrated load given in Table 205-1 whichever produces the greatest load effects. Unless otherwise specified the indicated concentration shall be assumed to be uniformly distributed over an area 750-mm square and shall be located so as to produce the maximum load effects in the structural member.

Provision shall be made in areas where vehicles are used or stored for concentrated loads, L , consisting of two or more loads spaced 1.5m nominally on center without uniform live loads. Each load shall be 40 percent of the gross weight of the maximum size vehicle to be accommodated. Parking garages for the storage of private or pleasure-type motor vehicles with no repair or refueling shall have a floor system designed for a concentrated load of not less than 9 kN acting on an area of 0.015 m² without uniform live loads. The condition of concentrated or uniform live load, combined in accordance with Section 203.3 or 203.4 as appropriate, producing the greatest stresses shall govern.

205.3.4 Special Loads

Provision shall be made for the special vertical and lateral loads as set forth in Table 205-2.

Table 205-1 Minimum Uniform and Concentrated Live Loads

Use or Occupancy		Uniform Load ¹	Concentrated Load
Category	Description	kPa	kN
1. Access floor systems	Office use	2.4	9.0 ²
	Computer use	4.8	9.0 ²
2. Armories	--	7.2	0
3. Theaters, assembly areas ³ and auditoriums	Fixed seats	2.9	0
	Movable seats	4.8	0
	Lobbies and platforms	4.8	0
	Stage areas	7.2	0
4. Bowling alleys, poolrooms and similar recreational areas	--	3.6	0
5. Catwalk for maintenance access	--	1.9	1.3
6. Cornices and marquees	--	3.6 ⁴	0
7. Dining rooms and restaurants	--	4.8	0
8. Exit facilities ⁵	--	4.8	0 ⁶
9. Parking Garages and Ramps	General storage and/or repair	4.8	-- ⁷
	Public parking and ramps	4.8	-- ⁷
	Private (residential) or pleasure-type motor vehicle storage	2.4	-- ⁷
10. Hospitals	Wards and rooms	1.9	4.5 ²
	Laboratories and operating rooms	2.9	4.5 ²
	Corridors above ground floor	3.8	4.5
11. Libraries	Reading rooms	2.9	4.5 ²
	Stack rooms	7.2	4.5 ²
	Corridors above ground floor	3.8	4.5
12. Manufacturing	Light	6.0	9.0 ²
	Heavy	12.0	13.4 ²
	Building corridors above ground floor	3.8	9.0

Table 205-1 Minimum Uniform and Concentrated Live Loads (*continued*)

Use or Occupancy		Uniform Load ¹	Concentrated Load
Category	Description	kPa	kN
13. Office	Call centers and business processing offices	2.9	9.0
	Lobbies and ground floor corridors	4.8	9.0
	Other offices	2.4	9.0 ²
14. Printing plants	Press rooms	7.2	11.0 ²
	Composing and linotype rooms	4.8	9.0 ²
15. Residential ⁸	Basic floor area	1.9	0 ⁶
	Exterior balconies	2.9 ⁴	0
	Decks	1.9 ⁴	0
	Storage	1.9	0
16. Restrooms ⁹	--	--	--
17. Reviewing stands, grandstands, bleachers, and folding and telescoping seating	--	4.8	0
18. Roof decks	Same as area served or occupancy	--	--
19. Schools	Classrooms	1.9	4.5 ²
	Corridors above ground floor	3.8	4.5
	Ground floor corridors	4.8	4.5
20. Sidewalks and driveways	Public access	12.0	-- ⁷
21. Storage	Light	6.0	--
	Heavy	12.0	--
22. Stores	Retail	4.8	4.5 ²
	Wholesale	6.0	13.4 ²
23. Pedestrian bridges and walkways	--	4.8	--

Notes for Table 205-1

¹ See Section 205.5 for live load reductions.² See Section 205.3.3, first paragraph, for area of load application.³ Assembly areas include such occupancies as dance halls, drill rooms, gymnasiums, playgrounds, plazas, terraces and similar occupancies that are generally accessible to the public.⁴ For special-purpose roofs, see Section 205.4.4.⁵ Exit facilities shall include such uses as corridors serving an occupant load of 10 or more persons, exterior exit balconies, stairways, fire escapes and similar uses.⁶ Individual stair treads shall be designed to support a 1.3 N concentrated load placed in a position that would cause maximum stress. Stair stringers may be designed for the uniform load set forth in the table.⁷ See Section 205.3.3, second paragraph, for concentrated loads. See Table 205-2 for vehicle barriers.⁸ Residential occupancies include private dwellings, apartments and hotel guest rooms.⁹ Restroom loads shall not be less than the load for the occupancy with which they are associated, but need not exceed 2.4 kPa.

Table 205-2 Special Loads¹

Use or Occupancy		Vertical Load	Lateral Load
Category	Description	kPa	kPa
1. Construction, public access at site (live load)	Walkway	7.2	-
	Canopy	7.2	-
2. Grandstands, reviewing stands, bleachers, and folding and telescoping seating (live load)	Seats and footboards	1.75 ²	See Note 3
3. Stage accessories (live load)	Catwalks	1.9	-
	Follow spot, projection and control rooms	2.4	-
4. Ceiling framing (live load)	Over stages	1.0	-
	All uses except over stages	0.5 ⁴	-
5. Partitions and interior walls,	-	-	0.25
6. Elevators and dumbwaiters (dead and live loads)	-	2*total load	-
7. Cranes (dead and live loads)	Total load including impact increase	1.25*total load ⁵	0.10*total load ⁶
8. Balcony railings and guardrails	Exit facilities serving an occupant load greater than 50 persons	-	0.75 kN/m ⁷
	Other than exit facilities	-	0.30 kN/m ⁷
	Components	-	1.2 ⁸
9. Vehicle barriers	-	-	27 kN ⁹
10. Handrails	-	See Note 10	See Note 10
11. Storage racks	Over 2.4 m high	Total loads ¹¹	See Table 208-13
12. Fire sprinkler structural support	-	1.1kN plus weight of water-filled pipe ¹²	See Table 208-13

Notes for Table 205-2

¹ The tabulated loads are minimum loads. Where other vertical loads required by the design would cause greater stresses, they shall be used. Loads are in kPa unless otherwise indicated in the table.

² Units is kN/m.

³ Lateral sway bracing loads of 350 N/m parallel and 145 N/m perpendicular to seat and footboards.

⁴ Does not apply to ceilings that have sufficient access from below, such that access is not required within the space above the ceiling. Does not apply to ceilings if the attic areas above the ceiling are not provided with access. This live load need not be considered as acting simultaneously with other live loads imposed upon the ceiling framing or its supporting structure.

⁵ The impact factors included are for cranes with steel wheels riding on steel rails. They may be modified if substantiating technical data acceptable to the building official is submitted. Live loads on crane support girders and their connections shall be taken as the maximum crane wheel loads. For pendant-operated traveling crane support girders and their connections, the impact factors shall be 1.10.

⁶ This applies in the direction parallel to the runway rails (longitudinal). The factor for forces perpendicular to the rail is 0.20 * the transverse traveling loads (trolley, cab, hooks and lifted loads). Forces shall be applied at top of rail and may be distributed among rails of multiple rail cranes and shall be distributed with due regard for lateral stiffness of the structures supporting these rails.

⁷ A load per lineal meter (kN/m) to be applied horizontally at right angles to the top rail.

⁸ Intermediate rails, panel fillers and their connections shall be capable of withstanding a load of 1.2 kPa applied horizontally at right angles over the entire tributary area, including openings and spaces between rails. Reactions due to this loading need not be combined with those of Note 7.

⁹ A horizontal load applied at right angles to the vehicle barrier at a height of 450 mm above the parking surface. The force may be distributed over a 300-mm square.

¹⁰ The mounting of handrails shall be such that the completed handrail and supporting structure are capable of withstanding a load of at least 890 N applied in any direction at any point on the rail. These loads shall not be assumed to act cumulatively with Note 9.

¹¹ Vertical members of storage racks shall be protected from impact forces of operating equipment, or racks shall be designed so that failure of one vertical member will not cause collapse of more than the bay or bays directly supported by that member.

¹² The 1.1-kN load is to be applied to any single fire sprinkler support point but not simultaneously to all support joints.

205.4 Roof Live Loads

205.4.1 General

Roofs shall be designed for the unit live loads, L_r , set forth in Table 205-3. The live loads shall be assumed to act vertically upon the area projected on a horizontal plane.

205.4.2 Distribution of Loads

Where uniform roof loads are involved in the design of structural members arranged to create continuity, consideration may be limited to full dead loads on all spans in combination with full roof live loads on adjacent spans and on alternate spans.

Exception:

Alternate span loading need not be considered where the uniform roof live load is 1.0 kPa or more.

For those conditions where light-gage metal preformed structural sheets serve as the support and finish of roofs, roof structural members arranged to create continuity shall be considered adequate if designed for full dead loads on all spans in combination with the most critical one of the following superimposed loads:

1. The uniform roof live load, L_r , set forth in Table 205-3 on all spans.
2. A concentrated gravity load, L_r , of 9 kN placed on any span supporting a tributary area greater than 18 m² to create maximum stresses in the member, whenever this loading creates greater stresses than those caused by the uniform live load. The concentrated load shall be placed on the member over a length of 0.75 m along the span. The concentrated load need not be applied to more than one span simultaneously.
3. Water accumulation as prescribed in Section 206.7.

205.4.3 Unbalanced Loading

Unbalanced loads shall be used where such loading will result in larger members or connections. Trusses and arches shall be designed to resist the stresses caused by unit live loads on one-half of the span if such loading results in reverse stresses, or stresses greater in any portion than the stresses produced by the required unit live load on the entire span. For roofs whose structures are composed of a stressed shell, framed or solid, wherein stresses caused by any point loading are distributed throughout the area of the shell, the requirements for unbalanced unit live load design may be reduced 50 percent.

205.4.4 Special Roof Loads

Roofs to be used for special purposes shall be designed for appropriate loads as approved by the building official. Greenhouse roof bars, purlins and rafters shall be designed to carry a 0.45 kN concentrated load, L_r , in addition to the uniform live load

205.5 Reduction of Live Loads

The design live load determined using the unit live loads as set forth in Table 205-1 for floors and Table 205-3, Method 2, for roofs may be reduced on any member supporting more than 15 m², including flat slabs, except for floors in places of public assembly and for live loads greater than 4.8 kPa, in accordance with the following equation:

$$R = r(A - 15) \quad (205-1)$$

The reduction shall not exceed 40 percent for members receiving load from one level only, 60 percent for other members or R , as determined by the following equation:

$$R = 23.1(1 + D/L) \quad (205-2)$$

where

- A = area of floor or roof supported by the member, m²
- D = dead load per square meter of area supported by the member, kPa
- L = unit live load per square meter of area supported by the member, kPa
- R = reduction in percentage,
- r = rate of reduction equal to 0.08 for floors. See Table 205-3 for roofs

For storage loads exceeding 4.8 kPa, no reduction shall be made, except that design live loads on columns may be reduced 20 percent.

The live load reduction shall not exceed 40 percent in garages for the storage of private pleasure cars having a capacity of not more than nine passengers per vehicle.

205.6 Alternate Floor Live Load Reduction

As an alternate to Equation 205-1, the unit live loads set forth in Table 205-1 may be reduced in accordance with Equation 205-3 on any member, including flat slabs, having an influence area of 40 m² or more.

$$L = L_o \left[0.25 + 4.57 \left(\frac{1}{\sqrt{A_I}} \right) \right] \quad (205-3)$$

where

A_I = influence area, m²

L = reduced design live load per square meter of area supported by the member

L_o = unreduced design live load per square meter of area supported by the member (Table 205-1)

The influence area A_I is four times the tributary area for a column, two times the tributary area for a beam, equal to the panel area for a two-way slab, and equal to the product of the span and the full flange width for a precast T-beam.

The reduced live load shall not be less than 50 percent of the unit live load L_o for members receiving load from one level only, nor less than 40 percent of the unit live load L_o for other members.

Table 205-3 Minimum Roof Live Loads ¹

Table 205-5 Minimum Roof Live Loads

ROOF SLOPE	Method 1			Method 2		
	Tributary Area (m ²)			Uniform Load ² (kPa)	Rate of Reduction, <i>r</i>	Maximum Reduction <i>R</i> (percentage)
	0 to 20	20 to 60	Over 60			
	Uniform Load (kPa)					
1. Flat ³ or rise less than 1-unit vertical in 3-unit horizontal (33.3% slope). Arch and dome with rise less than 1/8 of span.	1.00	0.75	0.60	1.00	0.08	40
2. Rise 1-unit vertical to less than 3-unit vertical in 3-unit horizontal (33.3% to less than 100% slope). Arch and dome with rise 1/8 of span to less than 3/8 of span.	0.75	0.70	0.60	0.75	0.06	25
3. Rise 1-unit vertical in 1-unit horizontal (100% slope) and greater. Arch or dome with rise 3/8 of span or greater.	0.60	0.60	0.60	0.60	<i>No reduction permitted</i>	
4. Awnings except cloth covered. ⁴	0.25	0.25	0.25	0.25		
5. Greenhouses, lath houses and agricultural buildings. ⁵	0.50	0.50	0.50	0.50		

¹ For special-purpose roofs, see Section 205.4.4.

² See Sections 205.5 and 205.6 for live-load reductions. The rate of reduction r in Equation 205-1 shall be as indicated in the table. The maximum reduction, R , shall not exceed the value indicated in the table.

³ A flat roof is any roof with a slope less than 1-unit vertical in 48-unit horizontal (2% slope). The live load for flat roofs is in addition to the ponding load required by Section 206.7.

⁴ See definition in Section 202.

⁵ See Section 205.4.4 for concentrated load requirements for greenhouse roof members.

SECTION 206 OTHER MINIMUM LOADS

206.1 General

In addition to the other design loads specified in this chapter, structures shall be designed to resist the loads specified in this section and the special loads set forth in Table 205-2. See Section 207 for design wind loads and Section 208 for design earthquake loads.

206.2 Other Loads

Buildings and other structures and portions thereof shall be designed to resist all loads due to applicable fluid pressures, F , lateral soil pressures, H , ponding loads, P , and self-straining forces, T . See Section 206.7 for ponding loads for roofs.

206.3 Impact Loads

The live loads specified in Sections 205.3 shall be assumed to include allowance for ordinary impact conditions. Provisions shall be made in the structural design for uses and loads that involve unusual vibration and impact forces. See Section 206.9.3 for impact loads for cranes, and Section 206.10 for heliport and helistop landing areas.

206.3.1 Elevators

All elevator loads shall be increased by 100% for impact.

206.3.2 Machinery

For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact:

1. Elevator machinery	100%
2. Light machinery, shaft- or motor-driven	20%
3. Reciprocating machinery or power-driven units	50%
4. Hangers for floors and balconies	33%

All percentages shall be increased where specified by the manufacturer.

206.4 Anchorage of Concrete and Masonry Walls

Concrete and masonry walls shall be anchored as required by Section 104.3.3. Such anchorage shall be capable of resisting the load combinations of Section 203.3 or 203.4 using the greater of the wind or earthquake loads required by this chapter or a minimum horizontal force of 4 kN/m of wall, substituted for E .

206.5 Interior Wall Loads

Interior walls, permanent partitions and temporary partitions that exceed 1.8 m in height shall be designed to resist all loads to which they are subjected but not less than a load, L , of 0.25 kPa applied perpendicular to the walls. The 0.25 kPa load need not be applied simultaneously with wind or seismic loads. The deflection of such walls under a load of 0.25 kPa shall not exceed 1/240 of the span for walls with brittle finishes and 1/120 of the span for walls with flexible finishes. See Table 208-13 for earthquake design requirements where such requirements are more restrictive.

Exception:

Flexible, folding or portable partitions are not required to meet the load and deflection criteria but must be anchored to the supporting structure to meet the provisions of this code.

206.6 Retaining Walls

Retaining walls shall be designed to resist loads due to the lateral pressure of retained material in accordance with accepted engineering practice. Walls retaining drained soil, where the surface of the retained soil is level, shall be designed for a load, H , equivalent to that exerted by a fluid weighing not less than 4.7 kPa per meter of depth and having a depth equal to that of the retained soil. Any surcharge shall be in addition to the equivalent fluid pressure.

Retaining walls shall be designed to resist sliding by at least 1.5 times the lateral force and overturning by at least 1.5 times the overturning moment, using allowable stress design loads.

206.7 Water Accumulation

All roofs shall be designed with sufficient slope or camber to ensure adequate drainage after the long-term deflection from dead load or shall be designed to resist ponding load, P , combined in accordance with Section 203.3 or 203.4. Ponding load shall include water accumulation from any source due to deflection.

206.8 Uplift on Floors and Foundations

In the design of basement floors and similar approximately horizontal elements below grade, the upward pressure of water, where applicable, shall be taken as the full hydrostatic pressure applied over the entire area. The hydrostatic load shall be measured from the underside of the construction. Any other upward loads shall be included in the design.

Where expansive soils are present under foundations or slabs-on-ground, the foundations, slabs, and other components shall be designed to tolerate the movement or resist the upward loads caused by the expansive soils, or the expansive soil shall be removed or stabilized around and beneath the structure.

206.9 Crane Loads

206.9.1 General

The crane load shall be the rated capacity of the crane. Design loads for the runway beams, including connections and support brackets of moving bridge cranes and monorail cranes shall include the maximum wheel loads of the crane and the vertical impact, lateral, and longitudinal forces induced by the moving crane.

206.9.2 Maximum Wheel Load

The maximum wheel loads shall be the wheel loads produced by the weight of the bridge, as applicable, plus the sum of the rated capacity and the weight of the trolley with the trolley positioned on its runway where the resulting load effect is maximum.

206.9.3 Vertical Impact Force

The maximum wheel loads of the crane shall be increased by the percentages shown below to determine the induced vertical impact or vibration force:

- | | |
|---|-----|
| 1. Monorail cranes (powered) | 25% |
| 2. Cab-operated or remotely operated bridge cranes (powered) | 25% |
| 3. Pendant-operated bridge cranes (powered) | 10% |
| 4. Bridge cranes or monorail cranes with hand-gear ridge, trolley and hoist | 0% |

206.9.4 Lateral Force

The lateral force on crane runway beams with electrically powered trolleys shall be calculated as 20% of the sum of the rated capacity of the crane and the weight of the hoist and trolley. The lateral force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction perpendicular to the beam, and shall be distributed with due regard to the lateral stiffness of the runway beam and supporting structure.

206.9.5 Longitudinal Forces

The longitudinal force on crane runway beams, except for bridge cranes with hand-gear bridges, shall be calculated as 10% of the maximum wheel loads of the crane. The longitudinal force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction parallel to the beam.

206.10 Heliport and Helistop Landing Areas

In addition to other design requirements of this chapter, heliport and helistop landing or touchdown areas shall be designed for the following loads, combined in accordance with Section 203.3 or 203.4:

1. Dead load plus actual weight of the helicopter.
2. Dead load plus a single concentrated impact load, L , covering 0.10 m^2 of 0.75 times the fully loaded weight of the helicopter if it is equipped with hydraulic-type shock absorbers, or 1.5 times the fully loaded weight of the helicopter if it is equipped with a rigid or skid-type landing gear.

The dead load plus a uniform live load, L , of 4.8 kPa. The required live load may be reduced in accordance with Section 205.5 or 205.6.

SECTION 207 WIND LOADS

207.1 Specifications

Buildings and other vertical structures shall be designed and constructed to resist wind loads as specified and presented in Sections 207A through 207F.

Antenna towers and antenna supporting structures shall be designed and constructed to resist wind loads as specified and presented in ANSI/TIA-222-G-2005, entitled as "Structural Standards for Steel Antenna Towers and Antenna Supporting Structures" and ANSI/TIA-222-G-1-2007, entitled as "Structural Standards for Steel Antenna Towers and Antenna Supporting Structures – Addendum 1."

207A General Requirements

Commentary:

The format and layout of the wind load provisions in this code have been significantly revised from NSCP 2001 and NSCP 2010 editions. The goal was to improve the organization, clarity, and use of the wind load provisions by creating individual sub-sections organized according to the applicable major subject areas. The wind load provisions are now presented in Sections 207A through 207F as opposed to prior editions, where the provisions were contained in a single section.

- Section 207A provides the basic wind design parameters that are applicable to the various wind load determination methodologies outlined in Sections 207B through 207F. Items covered in Section 207A include definitions, basic wind speed, exposure categories, internal pressures, enclosure classification, gust-effects, and topographic factors, among others. A general description of each section is provided below:
- Section 207B discusses about Directional Procedure for Enclosed, Partially Enclosed, and Open Buildings of All Heights: The procedure is the former "buildings of all heights method" in NSCP 2010 (ASCE 7-05), Method 2. A simplified procedure, based on the Directional Procedure, is provided for buildings up to 48m in height.
- Section 207C discusses about Envelope Procedure for Enclosed and Partially Enclosed Low-Rise Buildings: This procedure is the former "low-rise buildings method" in NSCP 2010 (ASCE 7-05) Method 2. This section also incorporates NSCP 2010 (ASCE 7-05) Method 1 for MWFRS applicable to the MWFRS of

enclosed simple diaphragm buildings less than 18 m in height.

- Section 207D discusses Other Structures and Building Appurtenances: A single section is dedicated to determining wind loads on non-building structures such as signs, rooftop structures, and towers.
- Section 207E discusses about Components and Cladding: This code addresses the determination of component and cladding loads in a single section. Analytical and simplified methods are provided based on the building height. Provisions for open buildings and building appurtenances are also addressed.
- Section 207F discusses about Wind Tunnel Procedure.

207A.1 Procedures

207A.1.1 Scope

Buildings and other structures, including the Main Wind-Force Resisting System (MWFRS) and all components and cladding (C&C) thereof, shall be designed and constructed to resist the wind loads determined in accordance with Section 207A through 207F. The provisions of this section define basic wind parameters for use with other provisions contained in this code.

Commentary:

The procedures specified in this code provide wind pressures and forces for the design of MWFRS and for the design of components and cladding (C&C) of buildings and other structures. The procedures involve the determination of wind directionality and velocity pressure, the selection or determination of an appropriate gust effect factor, and the selection of appropriate pressure or force coefficients. The procedure allows for the level of structural reliability required, the effects of differing wind exposures, the speed-up effects of certain topographic features such as hills and escarpments, and the size and geometry of the building or other structure under consideration. The procedure differentiates between rigid and flexible buildings and other structures, and the results generally envelop the most critical load conditions for the design of MWFRS as well as C&C.

The pressure and force coefficients provided in Sections 207B, 207C, 207D, and 207E have been assembled from the latest boundary-layer wind-tunnel and full-scale tests and from previously available literature. Because the boundary-layer wind-tunnel results were obtained for specific types of building, such as low- or high-rise buildings and buildings having specific types of structural

framing systems, the designer is cautioned against indiscriminate interchange of values among the figures and tables.

207A.1.2 Permitted Procedures

The design wind loads for buildings and other structures, including the MWFRS and C&C elements thereof, shall be determined using one of the procedures as specified in this article. An outline of the overall process for the determination of the wind loads, including section references, is provided in Figure 207A.1-1.

Commentary:

The design wind loads for buildings and other structures, including the MWFRS and C&C elements thereof, shall be determined using one of the procedures as specified in this article. An outline of the overall process for the determination of the wind loads, including section references, is provided in Figure 207A.1-1.

This version of the wind load standard provides several procedures (as illustrated in Table 207A.1-1) from which the designer can choose.

For MWFRS:

1. Directional Procedure for Buildings of All Heights (Section 207B)
2. Envelope Procedure for Low-Rise Buildings (Section 207C)
3. Directional Procedure for Building Appurtenances and Other Structures (Section 207D)
4. Wind Tunnel Procedure for All Buildings and Other Structures (Section 207F)

For Components and Cladding:

1. Analytical Procedure for Buildings and Building Appurtenances (Section 207E)
2. Wind Tunnel Procedure for All Buildings and Other Structures (Section 207F)

A "simplified method" for which the designer can select wind pressures directly from a table without any calculation, when the building meets all the requirements for application of the method, is provided for designing buildings using the Directional Procedure (Section 207B, Part 2), the Envelope Procedure (Section

207C, Part 2) and the Analytical Procedure for Components and Cladding (Section 207E).

Limitations. The provisions given under Section 207A.1.2 apply to the majority of site locations and buildings and structures, but for some projects, these provisions may be inadequate. Examples of site locations and buildings and structures (or portions thereof) that may require other approved standards, special studies using applicable recognized literature pertaining to wind effects, or using the wind tunnel procedure of Section 207F include:

1. Site locations that have channeling effects or wakes from upwind obstructions. Channeling effects can be caused by topographic features (e.g., a mountain gorge) or buildings (e.g., a neighboring tall building or a cluster of tall buildings). Wakes can be caused by hills or by buildings or other structures.
2. Buildings with unusual or irregular geometric shape, including barrel vaults, and other buildings whose shape (in plan or vertical cross-section) differs significantly from the shapes in Figures 207B.4-1, 207B.4-2, 207B.4-7, 207C.4-1, and 207E.4-1 to 207E.4-7. Unusual or irregular geometric shapes include buildings with multiple setbacks, curved facades, or irregular plans resulting from significant indentations or projections, openings through the building, or multi-tower buildings connected by bridges.
3. Buildings with response characteristics that result in substantial vortex-induced and/or torsional dynamic effects, or dynamic effects resulting from aero-elastic instabilities such as flutter or galloping. Such dynamic effects are difficult to anticipate, being dependent on many factors, but should be considered when any one or more of the following apply:
 - i. The height of the building is over 120 m.
 - ii. The height of the building is greater than 4 times its minimum effective width B_{min} , as defined below.
 - iii. The lowest natural frequency of the building is less than $n_1 = 0.25$ Hz.
 - iv. The reduced velocity

$$\frac{\bar{V}_z}{n_1 B_{min}} > 5$$

where

$$\bar{z} = 0.6h$$

$$\bar{V}_z = \text{the mean hourly velocity at height } \bar{z}$$

The minimum effective width $B_{\min}$ is defined as the minimum value of $\sum h_i B_i / \sum h_i$ considering all wind directions. The summations are performed over the height of the building for each wind direction, h_i is the height above grade of level i , and B_i is the width at level i normal to the wind direction.

4. Bridges, cranes, electrical transmission lines, guyed masts, highway signs and lighting structures, telecommunication towers, and flagpoles.

When undertaking detailed studies of the dynamic response to wind forces, the fundamental frequencies of the structure in each direction under consideration should be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis, and not utilizing approximate equations based on height.

Shielding. Due to the lack of reliable analytical procedures for predicting the effects of shielding provided by buildings and other structures or by topographic features, reductions in velocity pressure due to shielding are not permitted under the provisions of this chapter. However, this does not preclude the determination of shielding effects and the corresponding reductions in velocity pressure by means of the wind tunnel procedure in Section 207F.

207A.1.2.1 Main Wind-Force Resisting System (MWFRS)

Wind loads for MWFRS shall be determined using one of the following procedures:

1. Directional Procedure for buildings of all heights as specified in Section 207B for buildings meeting the requirements specified therein;
2. Envelope Procedure for low-rise buildings as specified in Section 207C for buildings meeting the requirements specified therein;
3. Directional Procedure for Building Appurtenances (rooftop structures and rooftop equipment) and Other Structures (such as solid freestanding walls and solid freestanding signs, chimneys, tanks, open signs, lattice frameworks, and trussed towers) as specified in Section 207D; or

4. Wind Tunnel Procedure for all buildings and all other structures as specified in Section 207F.

207A.1.2.2 Components and Cladding

Wind loads on components and cladding on all buildings and other structures shall be designed using one of the following procedures:

1. Analytical Procedures provided in Parts 1 through 6, as appropriate, of Section 207E; or
2. Wind Tunnel Procedure as specified in Section 207F.

207A.2 Definitions

The following definitions apply to the provisions of Section 207:

APPROVED is an acceptable to the authority having jurisdiction.

BASIC WIND SPEED, V is a three-second gust speed at 10m above the ground in Exposure C (see Section 207A.7.3) as determined in accordance with Section 207A.5.1.

BUILDING, ENCLOSED is a building that does not comply with the requirements for open or partially enclosed buildings.

BUILDING ENVELOPE is a cladding, roofing, exterior walls, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.

BUILDING AND OTHER STRUCTURE, FLEXIBLE are slender buildings and other structures that have a fundamental natural frequency less than 1 Hz.

BUILDING, LOW-RISE are enclosed or partially enclosed buildings that comply with the following conditions:

1. Mean roof height h less than or equal to 18m.
2. Mean roof height h does not exceed least horizontal dimension.

BUILDING, OPEN is a building having each wall at least 80 percent open. This condition is expressed for each wall by the equation $A_o \geq 0.8A_g$.

where:

A_o = total area of openings in a wall that receives positive external pressure, in m^2

A_g = the gross area of that wall in which A_o is identified, in m^2

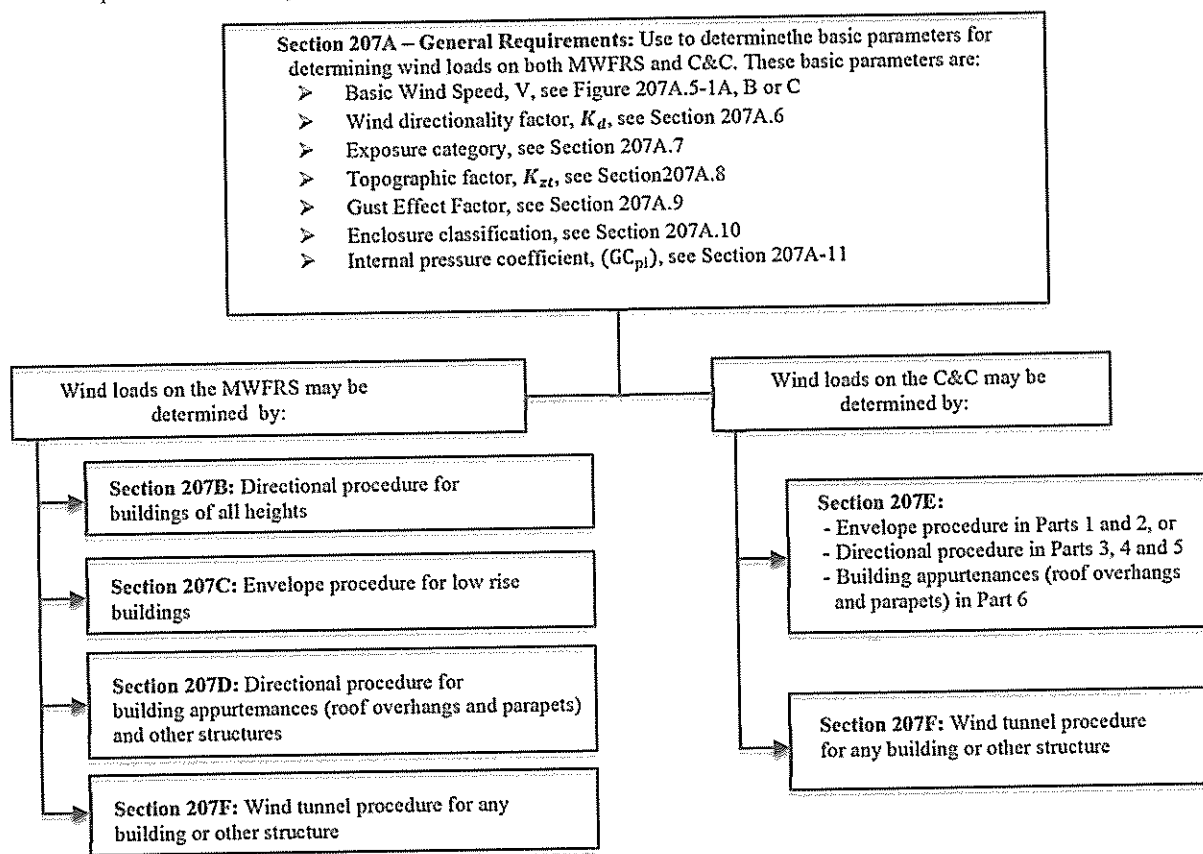


Figure 207A.1-1

Outline of Process for Determining Wind Loads. Additional Outlines and User Notes are Provided at the Beginning of each Chapter for more Detailed Step-By-Step Procedures for Determining the Wind Loads

BUILDING, PARTIALLY ENCLOSED is a building that complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10 percent.
2. The total area of openings in a wall that receives positive external pressure exceeds $0.37 m^2$.

or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent.

These conditions are expressed by the following equations:

1. $A_o > 1.10A_{oi}$
2. $A_o > 0.37 m^2$ or $0.01A_g$, whichever is smaller, and $A_{oi}/A_{gi} \leq 0.20$

where:

A_o, A_g = are as defined for Open Building
 A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o , in m^2
 A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g , in m^2

Components can be part of the MWFRS when they act as shear walls or roof diaphragms, but they may also be loaded as individual components. The engineer needs to use appropriate loadings for design of components, which may require certain components to be designed for more than one type of loading, for example, long-span roof trusses should be designed for loads associated with MWFRS, and individual members of trusses should also be designed for component and cladding loads (*Mehta and Marshall 1998*). Examples of cladding include wall coverings, curtain walls, roof coverings, exterior windows (fixed and operable) and doors, and overhead doors.

DIAPHRAGM in wind load applications has been added in ASCE 7-10. This definition, for the case of untopped steel decks, differs somewhat from the definition used in Section 12.3 of ASCE 7-10 because diaphragms under wind loads are expected to remain essentially elastic.

EFFECTIVE WIND AREA, A is an effective wind area is the area of the building surface used to determine (GC_p). This area does not necessarily correspond to the area of the building surface contributing to the force being considered. Two cases arise. In the usual case, the effective wind area does correspond to the area tributary to the force component being considered. For example, for a cladding panel, the effective wind area may be equal to the total area of the panel. For a cladding fastener, the effective wind area is the area of cladding secured by a single fastener. A mullion may receive wind from several cladding panels. In this case, the effective wind area is the area associated with the wind load that is transferred to the mullion.

The second case arises where components such as roofing panels, wall studs, or roof trusses are spaced closely together. The area served by the component may become long and narrow. To better approximate the actual load distribution in such cases, the width of the effective wind area used to evaluate (GC_p) need not be taken as less than one-third the length of the area. This increase in effective wind area has the effect of reducing the average wind pressure acting on the component. Note, however, that this effective wind area should only be used in determining the (GC_p) in Figures 207E.4-1 through 207E.4-6 and 207E.4-8. The induced wind load should be applied over the actual area tributary to the component being considered.

For membrane roof systems, the effective wind area is the area of an insulation board (or deck panel if insulation is not used) if the boards are fully adhered (or the membrane is adhered directly to the deck). If the insulation boards or membrane are mechanically attached or partially adhered, the effective wind area is the area of the board or membrane secured by a single fastener or individual spot or row of adhesive.

For typical door and window systems supported on three or more sides, the effective wind area is the area of the door or window under consideration. For simple spanning doors (e.g., horizontal spanning section doors or coiling doors), large specialty constructed doors (e.g., aircraft hangar doors), and specialty constructed glazing systems, the effective wind area of each structural component composing the door or window system should be used in calculating the design wind pressure.

MAIN WIND-FORCE RESISTING SYSTEM (MWFRS) can consist of a structural frame or an assemblage of structural elements that work together to transfer wind loads acting on the entire structure to the ground. Structural elements such as cross-bracing, shear walls, roof trusses, and roof diaphragms are part of the Main Wind-Force Resisting System (MWFRS) when they assist in transferring overall loads (*Mehta and Marshall 1998*).

WIND-BORNE DEBRIS REGIONS are defined to alert the designer to areas requiring consideration of missile impact design. These areas are located within tropical cyclone prone regions where there is a high risk of glazing failure due to the impact of wind-borne debris.

207A.3 Symbols and Notations

The following symbols and notation apply only to the provisions of Section 207A through 207F:

A	=	effective wind area, in m^2
A_f	=	area of open buildings and other structures either normal to the wind direction or projected on a plane normal to the wind direction, in m^2
A_g	=	the gross area of that wall in which A_o is identified, in m^2
A_{gI}	=	the sum of the gross surface areas of the building envelope (walls and roof) not including A_g , in m^2
A_o	=	total area of openings in a wall that receives positive external pressure, in m^2
A_{oi}	=	the sum of the areas of openings in the building envelope (walls and roof) not including A_o , in m^2
A_{og}	=	total area of openings in the building envelope in m^2
A_s	=	gross area of the solid freestanding wall or solid sign, in m^2
a	=	width of pressure coefficient zone, in m

B	= horizontal dimension of building measured normal to wind direction, in m	h	= mean roof height of a building or height of other structure, except that eave height shall be used for roof angle θ less than or equal to 10° , in m
$\bar{b}$	= mean hourly wind speed factor in Equation 207A.9-16 from Table 207A.9-1	h_e	= roof eave height at a particular wall, or the average height if the eave varies along the wall
$\hat{b}$	= 3-s gust speed factor from Table 207A.9-1	h_p	= height to top of parapet in Figure 207B.6-4 and 207E.7-1
C_f	= force coefficient to be used in determination of wind loads for other structures	I_z	= intensity of turbulence from Equation 207A.9-7
C_N	= net pressure coefficient to be used in determination of wind loads for open buildings	K_1, K_2, K_3	= multipliers in Figure 207A.8-1 to obtain K_{zt}
C_p	= external pressure coefficient to be used in determination of wind loads for buildings	K_d	= wind directionality factor in Table 207A.6-1
c	= turbulence intensity factor in Equation 207A.9-7 from Table 207A.9-1	K_h	= velocity pressure exposure coefficient evaluated at height $z = h$
D	= diameter of a circular structure or member, in m	K_z	= velocity pressure exposure coefficient evaluated at height z
D'	= depth of protruding elements such as ribs and spoilers, in m	K_{zt}	= topographic factor as defined in Section 207A.8
F	= design wind force for other structures, in N	L	= horizontal dimension of a building measured parallel to the wind direction, in m
G	= gust-effect factor	L_h	= distance upwind of crest of hill or escarpment in Figure 207A.8-1 to where the difference in ground elevation is half the height of the hill or escarpment, in m
G_f	= gust-effect factor for MWFRS of flexible buildings and other structures	L_r	= horizontal dimension of return corner for a solid freestanding wall or solid sign from Figure 207D.4-1, in m
(GC_r)	= product of external pressure coefficient and gust-effect factor to be used in determination of wind loads for rooftop structures	L_z	= integral length scale of turbulence, in m
(GC_p)	= product of external pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings	ℓ	= integral length scale factor from Table 207A.9-1, m
(GC_{pf})	= product of the equivalent external pressure coefficient and gust-effect factor to be used in determination of wind loads for MWFRS of low-rise buildings	N_1	= reduced frequency from Equation 207A.9-14
(GC_{pi})	= product of internal pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings	n_a	= approximate lower bound natural frequency (Hz) from Section 207A.9.2
(GC_{pn})	= combined net pressure coefficient for a parapet	n_1	= fundamental natural frequency, Hz
g_Q	= peak factor for background response in Equations 207A.9-6 and 207A.9-10	p	= design pressure to be used in determination of wind loads for buildings, in N/m^2
g_R	= peak factor for resonant response in Equation 207A.9-10	P_L	= wind pressure acting on leeward face in Figure 207B.4-8, in N/m^2
g_v	= peak factor for wind response in Equations 207A.9-6 and 207A.9-10	p_{net}	= net design wind pressure from Equation 207E.5-1, in N/m^2
H	= height of hill or escarpment in Figure 207A.8-1, in m	p_{net10}	= net design wind pressure for Exposure B at $h = 10$ m and $I = 1.0$ from Figure 207E.5-1, in N/m^2
		p_p	= combined net pressure on a parapet from Equation 207B.4-5, in N/m^2
		p_s	= net design wind pressure from Equation 207C.6-1, in N/m^2

p_{s10}	= simplified design wind pressure for Exposure B at $h = 10$ m and $I = 1.0$ from Figure 207C.6-1, in N/m^2
P_w	= wind pressure acting on windward face in Figure 207B.4-8, in N/m^2
Q	= background response factor from Equation 207A.9-8
q	= velocity pressure, in N/m^2
q_h	= velocity pressure evaluated at height $z = h$, in N/m^2
q_i	= velocity pressure for internal pressure determination, in N/m^2
q_p	= velocity pressure at top of parapet, in N/m^2
q_z	= velocity pressure evaluated at height z above ground, in N/m^2
R	= resonant response factor from Equation 207A.9-12
R_h, R_L	= values from Equations 207A.9-15
R_i	= reduction factor from Equation 207A.11-1
R_n	= value from Equation 207A.9-13
s	= vertical dimension of the solid freestanding wall or solid sign from Figure 207D.4-1, in m
r	= rise-to-span ratio for arched roofs
v	= height-to-width ratio for solid sign
V	= basic wind speed obtained from Figure 207A.5-1A through 207A.5-1C, in m/s. The basic wind speed corresponds to a 3-s gust speed at 10 m above the ground in Exposure Category C
V_i	= unpartitioned internal volume, m^3
$\bar{V}_z$	= mean hourly wind speed at height $\bar{z}$ m/s
W	= width of building in Figures 207E.4-3 and 207E.4-5A and 207E.4-5B and width of span in Figures 207E.4-4 and 207E.4-6, in m
x	= distance upwind or downwind of crest in Figure 207A.8-1, in m
z	= height above ground level, in m
$\bar{z}$	= equivalent height of structure, in m
z_g	= nominal height of the atmospheric boundary layer used in this code. Values appear in Table 207A.9-1
z_{min}	= exposure constant from Table 207A.9-1
α	= 3-s gust-speed power law exponent from Table 207A.9-1
$\hat{\alpha}$	= reciprocal of α from Table 207A.9-1
$\bar{\alpha}$	= mean hourly wind-speed power law exponent in Equation 207A.9-16 from Table 207A.9-1

β	= damping ratio, percent critical for buildings or other structures
ϵ	= ratio of solid area to gross area for solid freestanding wall, solid sign, open sign, face of a trussed tower, or lattice structure
$\bar{\epsilon}$	= integral length scale power law exponent in Equation 207A.9-9 from Table 207A.9-1
λ	= adjustment factor for building height and exposure from Figures 207C.6-1 and 207E.5-1
η	= value used in Equation 207A.9-15 (see Section 207A.9.4)
θ	= angle of plane of roof from horizontal, in degrees

207A.4 General

207A.4.1 Sign Convention

Positive pressure acts toward the surface and negative pressure acts away from the surface.

207A.4.2 Critical Load Condition

Values of external and internal pressures shall be combined algebraically to determine the most critical load.

207A.4.3 Wind Pressures Acting on Opposite Faces of Each Building Surface

In the calculation of design wind loads for the MWFRS and for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be taken into account.

Commentary: See ASCE 7-10, Section 6.5.5.2.

Section 207A.4.3 is included in the code to ensure that internal and external pressures acting on a building surface are taken into account by determining a net pressure from the algebraic sum of those pressures. For additional information on the application of the net components and cladding wind pressure acting across a multilayered building envelope system, including air-permeable cladding, refer to Section C207E.1.5.

207A.5 Wind Hazard Map

207A.5.1 Basic Wind Speed

The basic wind speed, V , used in the determination of design wind loads on buildings and other structures shall be determined from Figure 207A.5-1 as follows, except as provided in Section 207A.5.2 and 207A.5.3:

For Occupancy Category III, IV and V buildings and other structures – use Figure 207A.5-1A.

For Occupancy Category II buildings and other structures – use Figure 207A.5-1B.

For Occupancy Category I buildings and other structures – use Figure 207A.5-1C.

The wind shall be assumed to come from any horizontal direction. The basic wind speed shall be increased where records or experience indicate that the wind speeds are higher than those reflected in Figure 207A.5-1.

Commentary:

This edition of NSCP departs from prior editions by providing wind maps that are directly applicable for determining pressures for strength design approaches. Rather than using a single map with importance factors and a load factor for each building occupancy category, in this edition there are different maps for different categories of building occupancies. The updated maps are based on a new and more complete analysis of tropical cyclone characteristics (Vickery et al. 2008a, 2008b and 2009) performed over the past 10 years.

The decision to move to multiple-strength design maps in conjunction with a wind load factor of 1.0 instead of using a single map used with an importance and a load factor of 1.6 relied on several factors important to an accurate wind specification:

1. *A strength design wind speed map brings the wind loading approach in line with that used for seismic loads in that they both essentially eliminate the use of a load factor for strength design.*

2. *Multiple maps remove inconsistencies in the use of importance factors that actually should vary with location and between tropical cyclone-prone and non-tropical cyclone-prone regions for Occupancy Category III, IV and V structures and acknowledge that the demarcation between tropical cyclone and non-tropical cyclone winds change with the recurrence interval.*
3. *The new maps establish uniformity in the return period for the design-basis winds, and they more clearly convey that information.*
4. *The new maps, by providing the design wind speed directly, more clearly inform owners and their consultants about the storm intensities for which designs are performed.*

Selection of Return Periods. *In the development of the design wind speed map used in Section 207 NSCP 2010, the Wind Load Subcommittee evaluated the wind importance factor, I_w , that had been in use since 1982. The task committee recognized that using a uniform value of the wind importance factor probably was not appropriate because risk varies with location along the coast.*

To determine the return periods to be used in the new mapping approach, the task committee needed to meet with PAGASA scientists, gather historical records and evaluate representative return periods for wind speeds determined in accordance with Section 207 NSCP 2010 and earlier, wherein determination of pressures appropriate for strength design started with mapped wind speeds, but involved multiplication by importance factors and a wind load factor to achieve pressures that were appropriate for strength design. Furthermore, it was assumed that the variability of the wind speed dominates the calculation of the wind load factor. The strength design wind load, W_T , is given as:

$$W_T = C_F(V_{50}I)^2W_{LF} \quad (C207A.5-1)$$

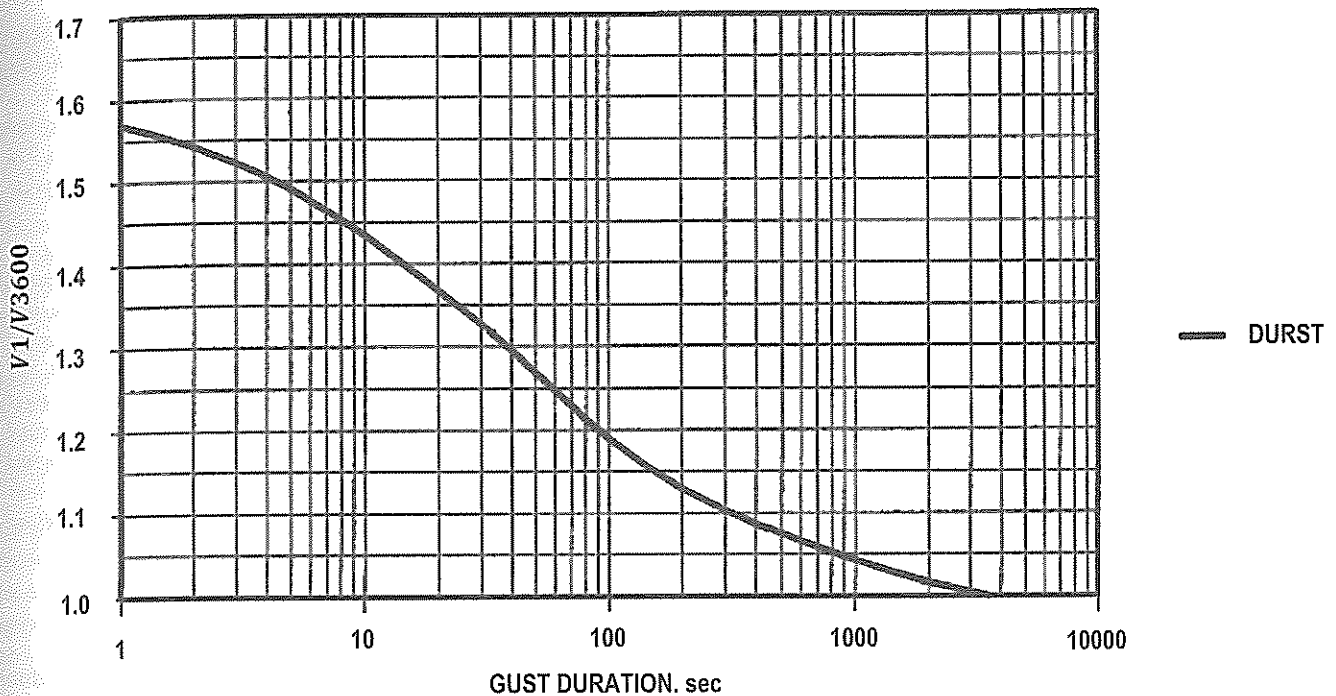


Figure C207A.5-1
Maximum Speed Averaged over t s to Hourly Mean Speed

where C_F is a building, component, or structure specific coefficient that includes the effects of things like building height, building geometry, terrain, and gust factor as computed using the procedures outlined in NSCP 2010. V_{50} is the 50-year return period design wind speed, W_{LF} is the wind load factor, and I is the importance factor.

Starting with the nominal return period of 50 years, the ratio of the wind speed for any return period to the 50-year return period wind speed can be computed from Peterka and Shahid (1998):

$$V_T/V_{50} = [0.36 + 0.11n(12T)] \quad (C207A.5-2)$$

where T is the return period in years and V_T is the T -year return period wind speed. The strength design wind load, W_T , occurs when:

$$W_T = C_F V_T^2 = C_F V_{50}^2 W_{LF} \quad (C207A.5-3)$$

Thus,

$$\begin{aligned} V_T/V_{50} &= [0.36 + 0.11n(12T)] \\ &= \sqrt{W_{LF}} \end{aligned} \quad (C207A.5-4)$$

and from Equation C207A.5-4, the return period T associated with the strength design wind speed is:

$$T = 0.00228 \exp(10\sqrt{W_{LF}}) \quad (C207A.5-5)$$

Using the wind load factor of 1.6 as specified in Section 207 NSCP 2010, from Equation C207A.5-5 we get $T = 709$ years, and therefore $V_{\text{design}} = V_{709}/\sqrt{W_{LF}} \approx V_{700}/\sqrt{W_{LF}}$. Thus for Occupancy Category IV structures, the basic wind speed is associated with a return period of 700 years, or an annual exceedance probability of 0.0014.

The importance factor used in Section 207 NSCP 2010 and earlier for the computation of wind loads for the design of Occupancy Category I and II structures is defined so that the nominal 50-year return period non-tropical cyclone wind speed is increased to be representative of a 100-year return period value. Following the approach used above to estimate the resulting effective strength design return period associated with a 50-year basic design speed, in the case of the 100-year return period basic wind speed in the non-tropical cyclone-prone regions, we find that:

$$T = 0.00228 \exp[10(V_{100}/V_{50})\sqrt{W_{LF}}] \quad (C207A.5-6)$$

where for V_{100}/V_{50} computed from Equation C207A.5-4 with $W_{LF} = 1.6$, we find $T = 1,697$ years. In the development of Equation C207A.5-6, the term $(V_{100}/V_{50})W_{LF}$ replaces the W_{LF} used in Equation C207A.5-5, effectively resulting in a higher load factor for Occupancy Category I, II and III structures equal to $W_{LF}(V_{100}/V_{50})^2$. Thus for Occupancy Category I and II structures, the basic wind speed is associated with a return period of 1,700 years, or an annual exceedance probability of 0.000588. Similarly, the 25-year return period wind speed associated with Occupancy Category III, IV and V buildings equates to a 300-year return period wind speed with a wind load factor of 1.0.

Wind Speeds. The wind speed maps of Figure 207A.5-1 present basic wind speeds for the entire archipelago of the Philippines. The wind speeds correspond to 3-sec gust speeds at 10 m above ground for exposure category C.

Serviceability Wind Speeds. For applications of serviceability, design using maximum likely events, or other applications, it may be desired to use wind speeds associated with mean recurrence intervals other than those given in Figures 207A.5-1A to 207A.5-1C. To accomplish this, previous editions of NSCP 2010 provided tables with factors that enabled the user to adjust the basic design wind speed (previously having a return period of 50 years to wind speeds associated with other return periods).

For applications of serviceability, design using maximum likely events, or other applications, Appendix C presents maps of peak gust wind speeds at 10 m above ground in Exposure C conditions for return periods of 10, 25, 50, and 100 years.

The probability P_n that the wind speed associated with a certain annual probability P_a will be equaled or exceeded at least once during an exposure period of n years is given by:

$$P_n = 1 - (1 - P_a)^n \quad (\text{C207A.5-7})$$

As an example, if a wind speed is based upon $P_a = 0.02$ (50-year mean recurrence interval), there exists a probability of 0.40 that this speed will be equaled or exceeded during a 25-year period, and a 0.64 probability of being equaled or exceeded in a 50-year period.

Similarly, if a wind speed is based upon $P_a = 0.00143$ (700-year mean recurrence interval), there exists a 3.5% probability that this speed will be equaled or exceeded during a 25-year period, and a 6.9% probability of being equaled or exceeded in a 50-year period.

Some products have been evaluated and test methods have been developed based on design wind speeds that are consistent with the unfactored load effects typically used in Allowable Stress Design. Table C207A.5-6 provides conversion from the strength design-based design wind speeds used in the ASCE 7-10 design wind speed maps and the Section 207 NSCP 2010 design wind speeds used in these product evaluation reports and test methods. A column of values is also provided to allow coordination with ASCE 7-93 design wind speeds.

207A.5.2 Special Wind Regions

Mountainous terrain, gorges, and special wind regions shown in Figure 207A.5-1 shall be examined for unusual wind conditions. The authority having jurisdiction shall, if necessary, adjust the values given in Figure 207A.5-1 to account for higher local wind speeds. Such adjustment shall be based on meteorological information and an estimate of the basic wind speed obtained in accordance with the provisions of Section 207A.5.3.

Commentary:

Although the wind speed map of Figure 207A.5-1 is valid for most regions of the country, there are special regions in which wind speed anomalies are known to exist. Some of these special regions are noted in Figure 207A.5-1. Winds blowing over mountain ranges or through gorges or river valleys in these special regions can develop speeds that are substantially higher than the values indicated on the map. When selecting basic wind speeds in these special regions, use of regional climatic data and consultation with a wind engineer or meteorologist is advised.

It is also possible that anomalies in wind speeds exist on a micrometeorological scale. For example, wind speed-up over hills and escarpments is addressed in Section 207A.8. Wind speeds over complex terrain may be better determined by wind-tunnel studies as described in Section 207F. Adjustments of wind speeds should be made at the micrometeorological scale on the basis of wind engineering or meteorological advice and used in accordance with the provisions of Section 207A.5.3 when such adjustments are warranted. Due to the complexity of mountainous terrain and valley gorges in Hawaii, there are topographic wind speed-up effects that cannot be

addressed solely by Figure 207A.8-1 (Applied Research Associates 2001).

207A.5.3 Estimation of Basic Wind Speeds from Regional Climatic Data

In areas outside tropical cyclone-prone regions, regional climatic data shall only be used in lieu of the basic wind speeds given in Figure 207A.5-1 when (1) approved extreme-value statistical-analysis procedures have been employed in reducing the data; and (2) the length of record, sampling error, averaging time, anemometer height, data quality, and terrain exposure of the anemometer have been taken into account. Reduction in basic wind speed below that of Figure 207A.5-1 shall be permitted.

In tropical cyclone-prone regions, wind speeds derived from simulation techniques shall only be used in lieu of the basic wind speeds given in Figure 207A.5-1 when approved simulation and extreme value statistical analysis procedures are used.

In areas outside tropical cyclone-prone regions, when the basic wind speed is estimated from regional climatic data, the basic wind speed shall not be less than the wind speed associated with the specified mean recurrence interval, and the estimate shall be adjusted for equivalence to a 3-s gust wind speed at 10m above ground in Exposure C. The data analysis shall be performed in accordance with this section.

Commentary:

When using regional climatic data in accordance with the provisions of Section 207A.5.3 and in lieu of the basic wind speeds given in Figure 207A.5-1, the user is cautioned that the gust factors, velocity pressure exposure coefficients, gust effect factors, pressure coefficients, and force coefficients of this code are intended for use with the 3-s gust speed at 10m above ground in open country. It is necessary, therefore, that regional climatic data based on a different averaging time, for example, hourly mean or fastest mile, be adjusted to reflect peak gust speeds at 10m above ground in open country.

In using local data, it should be emphasized that sampling errors can lead to large uncertainties in specification of the wind speed. Sampling errors are the errors associated with the limited size of the climatological data samples (years of record of annual extremes). It is possible to have a 8.9m/s error in wind speed at an individual station with a record length of 30 years. While local records of limited extent often must be used to define wind speeds in special wind areas, care and conservatism should be exercised in their use.

If meteorological data are used to justify a wind speed lower than 177-km/h 700-yr peak gust at 10 m, an analysis of sampling error is required to demonstrate that the wind record could not occur by chance. This can be accomplished by showing that the difference between predicted speed and 177 km/h contains two to three standard deviations of sampling error (Simiu and Scanlan 1996). Other equivalent methods may be used.

207A.5.4 Limitation

Tornadoes have not been considered in developing the basic wind-speed distributions.

207A.6 Wind Directionality

The wind directionality factor, K_d , shall be determined from Table 207A.6-1. This directionality factor shall only be included in determining wind loads when the load combinations specified in Sections 2.3 and 2.4 are used for the design. The effect of wind directionality in determining wind loads in accordance with Section 207F shall be based on an analysis for wind speeds that conforms to the requirements of Section 207A.5.3.

Commentary:

The wind load factor 1.3 in ASCE 7-95 included a "wind directionality factor" of 0.85 (Ellingwood 1981 and Ellingwood et al. 1982). This factor accounts for two effects: (1) The reduced probability of maximum winds coming from any given direction and (2) the reduced probability of the maximum pressure coefficient occurring for any given wind direction. The wind directionality factor (identified as K_d in the code) is tabulated in Table 207A.6-1 for different structure types. As new research becomes available, this factor can be directly modified. Values for the factor were established from references in the literature and collective committee judgment. The K_d value for round chimneys, tanks, and similar structures is given as 0.95 in recognition of the fact that the wind load resistance may not be exactly the same in all directions as implied by a value of 1.0. A value of 0.85 might be more appropriate if a triangular trussed frame is shrouded in a round cover. A value of 1.0 might be more appropriate for a round chimney having a lateral load resistance equal in all directions. The designer is cautioned by the footnote to Table 207A.6-1 and the statement in Section 207A.6, where reference is made to the fact that this factor is only to be used in conjunction with the load combinations specified in Sections 2.3 and 2.4 of ASCE 7-10.

207A.7 Exposure

For each wind direction considered, the upwind exposure shall be based on ground surface roughness that is determined from natural topography, vegetation, and constructed facilities.

Commentary:

The descriptions of the surface roughness categories and exposure categories in Section 207A.7 have been expressed as far as possible in easily understood verbal terms that are sufficiently precise for most practical applications. Upwind surface roughness conditions required for Exposures B and D are shown schematically in Figures C207A.7-1 and C207A.7-2, respectively. For cases where the designer wishes to make a more detailed assessment of the surface roughness category and exposure category, the following more mathematical description is offered for guidance (Irwin 2006). The ground surface roughness is best measured in terms of a roughness length parameter called z_0 . Each of the surface roughness categories B through D correspond to a range of values of this parameter, as does the even rougher category A used in previous versions of the code in heavily built-up urban areas but removed in the present edition. The range of z_0 in meters (m) for each terrain category is given in Table C207A.7-1. Exposure A has been included in Table C207A.7-1 as a reference that may be useful when using the Wind Tunnel Procedure. Further information on values of z_0 in different types of terrain can be found in Simiu and Scanlan (1996) and Table C207A.7-2 based on Davenport et al. (2000) and Wieringa et al. (2001). The roughness classifications in Table C207A.7-2 are not intended to replace the use of exposure categories as required in the code for structural design purposes. However, the terrain roughness classifications in Table C207A.7-2 may be related to ASCE 7 exposure categories by comparing z_0 values between Table C207A.7-1 and C207A.7-2. For example, the z_0 values for Classes 3 and 4 in Table C207A.7-2 fall within the range of z_0 values for Exposure C in Table C207A.7-1. Similarly, the z_0 values for Classes 5 and 6 in Table C207A.7-2 fall within the range of z_0 values for Exposure B in Table C207A.7-1.

Research described in Powell et al. (2003), Donelan et al. (2004), and Vickery et al. (2008b) showed that the drag coefficient over the ocean in high winds in tropical cyclones did not continue to increase with increasing wind speed as previously believed (e.g., Powell 1980). These studies showed that the sea surface drag coefficient, and hence the aerodynamic roughness of the ocean, reached a maximum at mean wind speeds of about 30m/s. There is some evidence that the drag

coefficient actually decreases (i.e., the sea surface becomes aerodynamically smoother) as the wind speed increases further (Powell et al. 2003) or as the tropical cyclone radius decreases (Vickery et al. 2008b). The consequences of these studies are that the surface roughness over the ocean in a tropical cyclone is consistent with that of exposure D rather than exposure C. Consequently, the use of exposure D along the tropical cyclone coastline is now required.

For Exposure B the tabulated values of K_z correspond to $z_0 = 0.2$ m, which is below the typical value of 0.3 m, whereas for Exposures C and D they correspond to the typical value of z_0 . The reason for the difference in Exposure B is that this category of terrain, which is applicable to suburban areas, often contains open patches, such as highways, parking lots, and playing fields. These cause local increases in the wind speeds at their edges. By using an exposure coefficient corresponding to a lower than typical value of z_0 , some allowance is made for this. The alternative would be to introduce a number of exceptions to use of Exposure B in suburban areas, which would add an undesirable level of complexity.

The value of z_0 for a particular terrain can be estimated from the typical dimensions of surface roughness elements and their spacing on the ground area using an empirical relationship, due to Lettau (1969), which is:

$$z_0 = 0.5 H_{ob} \frac{S_{ob}}{A_{ob}} \quad (\text{C207A.7-1})$$

- H_{ob} = the average height of the roughness in the upwind terrain
- S_{ob} = the average vertical frontal area per obstruction presented to the wind
- A_{ob} = the average area of ground occupied by each obstruction, including the open area surrounding it

Vertical frontal area is defined as the area of the projection of the obstruction onto a vertical plane normal to the wind direction. The area S_{ob} may be estimated by summing the approximate vertical frontal areas of all obstructions within a selected area of upwind fetch and dividing the sum by the number of obstructions in the area. The average height H_{ob} may be estimated in a similar way by averaging the individual heights rather than using the frontal areas. Likewise A_{ob} may be estimated by dividing the size of the selected area of upwind fetch by the number of obstructions in it.

As an example, if the upwind fetch consists primarily of single family homes with typical height $H_{ob} = 6\text{m}$, vertical frontal area (including some trees on each lot) of 100m^2 , and ground area per home of $1,000\text{m}^2$, then z_0 is calculated to be $z_0 = 0.5 \times 20 \times 100/1,000 = 0.3\text{m}$, which falls into exposure category B according to Table C207A.7-1.

Trees and bushes are porous and are deformed by strong winds, which reduce their effective frontal areas (ESDU, 1993). For conifers and other evergreens no more than 50 percent of their gross frontal area can be taken to be effective in obstructing the wind. For deciduous trees and bushes no more than 15 percent of their gross frontal area can be taken to be effective in obstructing the wind. Gross frontal area is defined in this context as the projection onto a vertical plane (normal to the wind) of the area enclosed by the envelope of the tree or bush.

Ho (1992) estimated that the majority of buildings (perhaps as much as 60 percent to 80 percent) have an exposure category corresponding to Exposure B. While the relatively simple definition in the code will normally suffice for most practical applications, oftentimes the designer is in need of additional information, particularly with regard to the effect of large openings or clearings (e.g., large parking lots, freeways, or tree clearings) in the otherwise "normal" ground surface roughness B. The following is offered as guidance for these situations:

The simple definition of Exposure B given in the body of the code, using the surface roughness category definition, is shown pictorially in Figure C207A.7-1. This definition applies for the surface roughness B condition prevailing 800 m upwind with insufficient "open patches" as defined in the following text to disqualify the use of Exposure B.

An opening in the surface roughness B large enough to have a significant effect on the exposure category

determination is defined as an "open patch." An open patch is defined as an opening greater than or equal to approximately 50 m on each side (i.e., greater than 50 m by 50 m). Openings smaller than this need not be considered in the determination of the exposure category.

The effect of open patches of surface roughness C or D on the use of exposure category B is shown pictorially in Figures C207A.7-3 and C207A.7-4. Note that the plan location of any open patch may have a different effect for different wind directions.

Aerial photographs, representative of each exposure type, are included in the commentary to aid the user in establishing the proper exposure for a given site. Obviously, the proper assessment of exposure is a matter of good engineering judgment. This fact is particularly true in light of the possibility that the exposure could change in one or more wind directions due to future demolition and/or development.

207A.7.1 Wind Directions and Sectors

For each selected wind direction at which the wind loads are to be determined, the exposure of the building or structure shall be determined for the two upwind sectors extending 45° either side of the selected wind direction. The exposure in these two sectors shall be determined in accordance with Sections 207A.7.2 and 207A.7.3, and the exposure whose use would result in the highest wind loads shall be used to represent the winds from that direction.



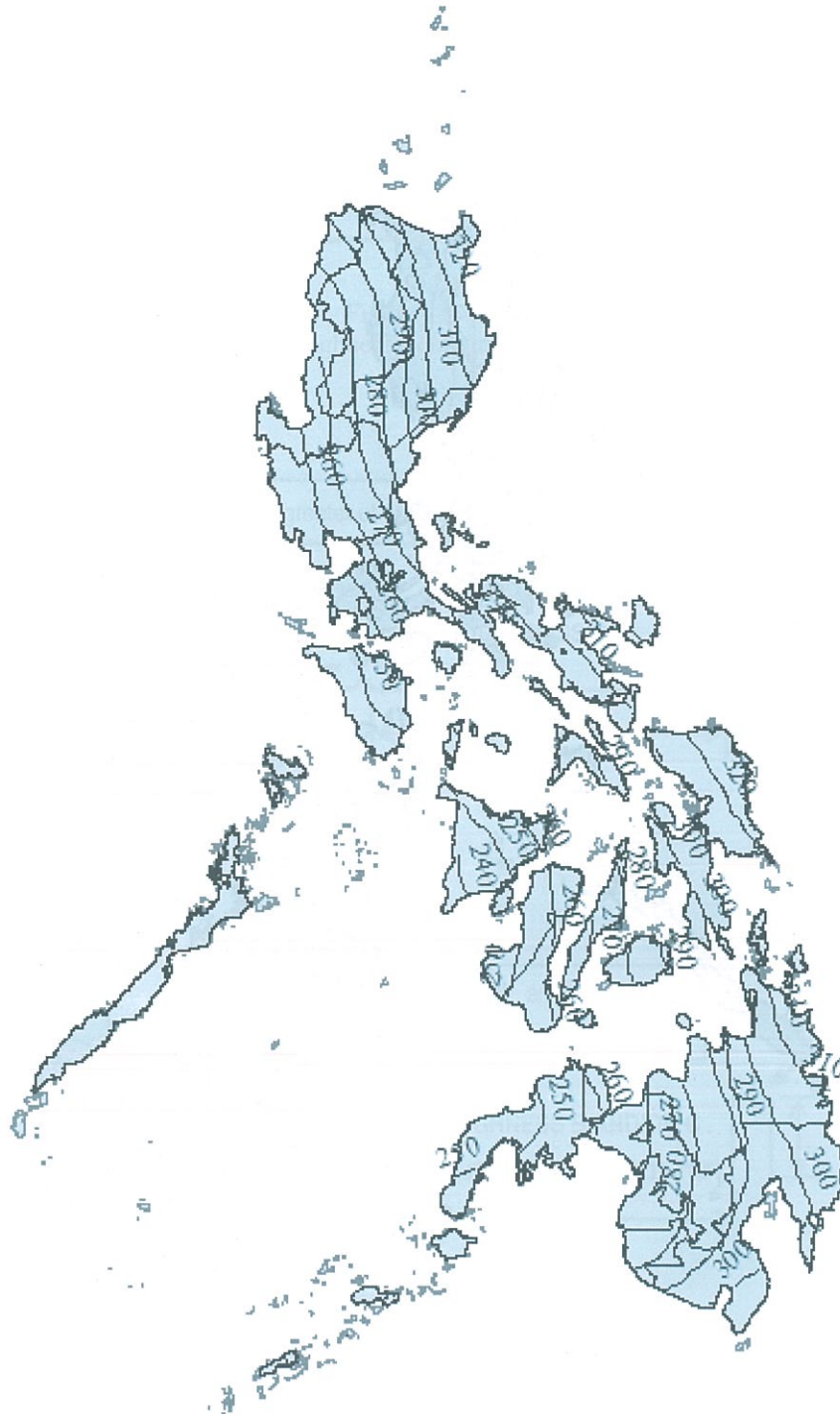
Notes:

1. Values are nominal design 3-second gust wind speeds in kilometers per hour at 10 m above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333, MRI = 300 years).
6. Results are from PAGASA.

Note:

- 1.
- 2.
- 3.
- 4.
- 5.
- 6.

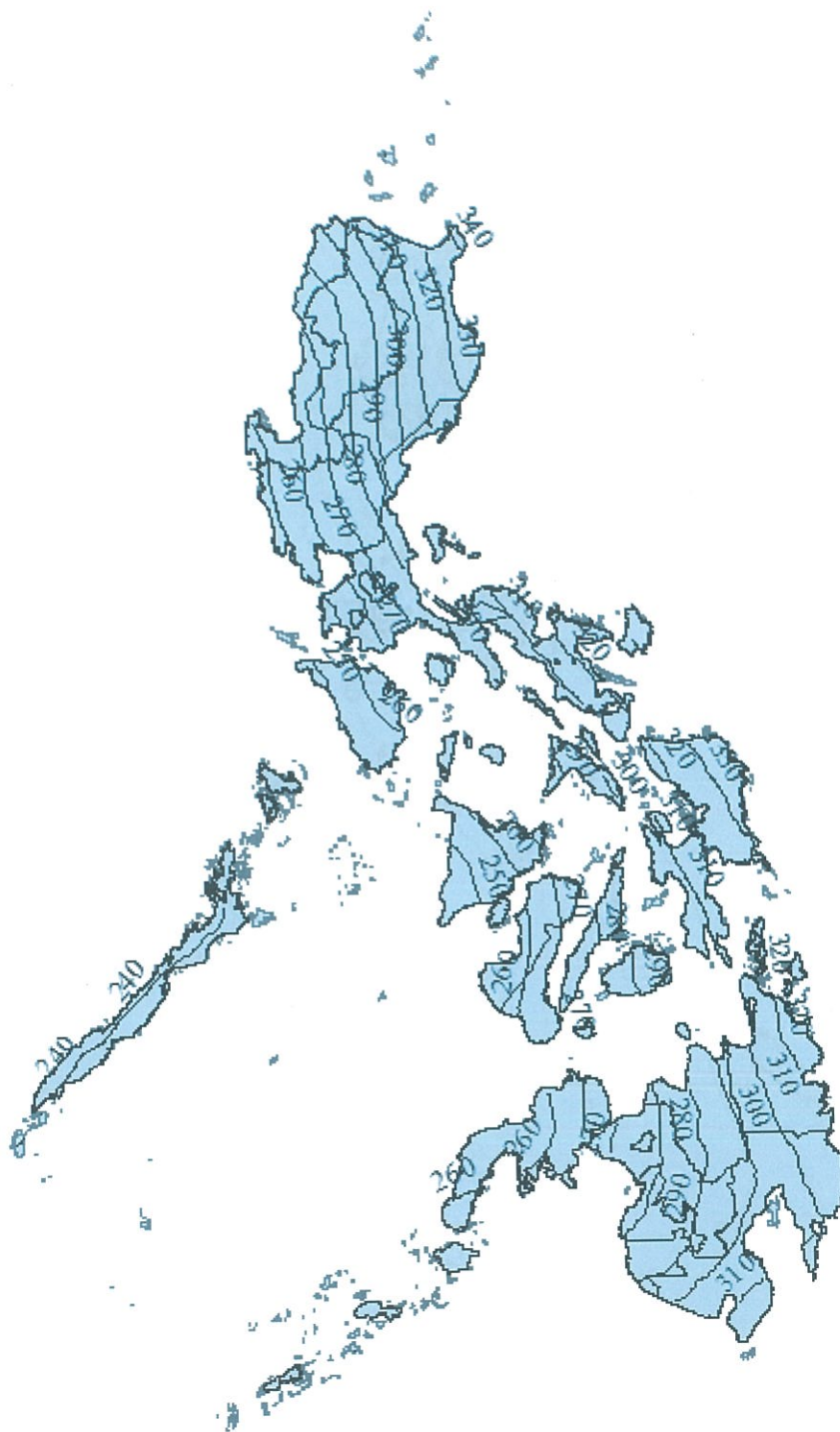
Figure 207A.5-1A Basic Wind Speeds for Occupancy Category III, IV and V Buildings and Other Structures



Notes:

1. Values are nominal design 3-second gust wind speeds in kilometers per hour at 10 m above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 years).
6. Results are from PAGASA.

Figure 207A.5-1B Basic Wind Speeds for Occupancy Category II Buildings and Other Structures
National Structural Code of the Philippines Volume I, 7th Edition, 2015



Notes:

1. Values are nominal design 3-second gust wind speeds in kilometers per hour at 10 m above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1700 years).
6. Results are from PAGASA.

Figure 207A.5-1C Basic Wind Speeds for Occupancy Category I Buildings and Other Structures

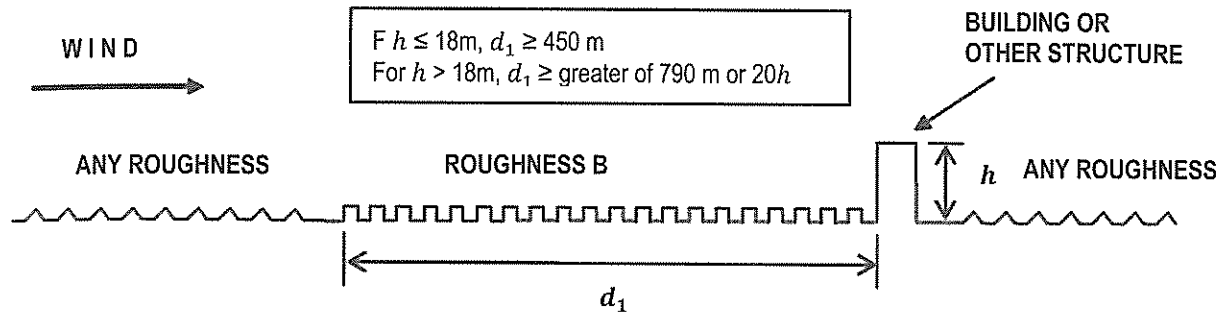
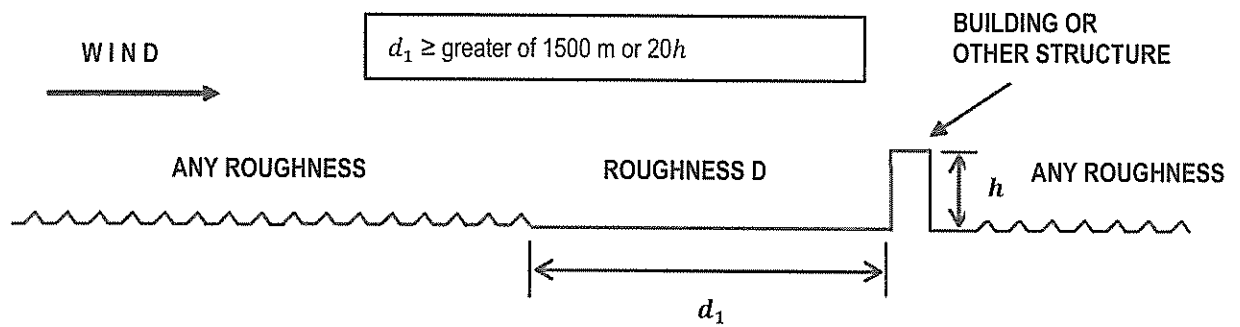
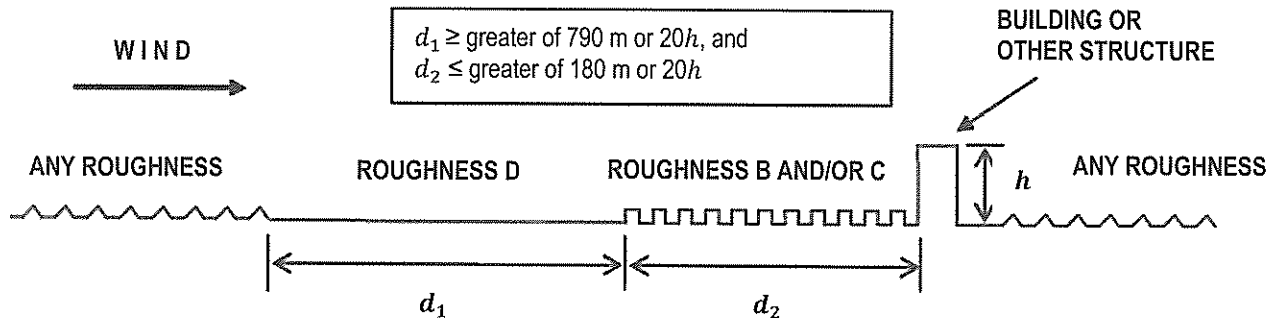


Figure C207A.7-1

Upwind Surface Roughness Conditions Required for Exposure B.



(a)



(b)

Figure C207A.7-2

Upwind Surface Roughness Conditions Required for Exposure D, for the Cases with
 (a) Surface Roughness D Immediately Upwind of the Building, and (b) Surface Roughness B and/or C Immediately Upwind of the Building

Table 207A.6-1
Wind Directionality Factor, K_d

Structure Type	Directionality Factor K_d^*
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid Freestanding Walls and Solid Freestanding and Attached Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

**Directionality Factor K_d has been calibrated with combinations of loads specified in Section 203. This factor shall only be applied when used in conjunction with load combinations specified in Sections 203.3 and 203.4.*

207A.7.2 Surface Roughness Categories

A ground Surface Roughness within each 45° sector shall be determined for a distance upwind of the site as defined in Section 207A.7.3 from the categories defined in the following text, for the purpose of assigning an exposure category as defined in Section 207A.7.3.

Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C: Open terrain with scattered obstructions having heights generally less than 9 m. This category includes flat open country and grasslands.

Surface Roughness D: Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats, and unbroken ice.

207A.7.3 Exposure Categories

Exposure B: For buildings with a mean roof height of less than or equal to 9 m, Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance greater than 450 m. For buildings with a mean roof height greater than 9 m, Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance greater than 790 m or 20 times the height of the building, whichever is greater.

Exposure C: Exposure C shall apply for all cases where Exposures B or D do not apply.

Exposure D: Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance greater than 1500 m or 20 times the building height, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is exposure B or C, and the site is within a distance of 180 m or 20 times the building height, whichever is greater, from

an Exposure D condition as defined in the previous sentence.

For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used.

Exception:

An intermediate exposure between the preceding categories is permitted in a transition zone provided that it is determined by a rational analysis method defined in the recognized literature.

207A.7.4 Exposure Requirements

Commentary:

The provision in Section 207A.5.1 requires that a structure be designed for winds from all directions. A rational procedure to determine directional wind loads is as follows. Wind load for buildings using Section 207B.4.1 and Figures 207B.4-1, 207B.4-2 or 207B.4-3 are determined for eight wind directions at 45° intervals, with four falling along primary building axes as shown in Figure C207A.7-5. For each of the eight directions, upwind exposure is determined for each of two 45° sectors, one on each side of the wind direction axis. The sector with the exposure giving highest loads will be used to define wind loads for that direction. For example, for winds from the north, the exposure from sector one or eight, whichever gives the higher load, is used. For wind from the east, the exposure from sector two or three, whichever gives the highest load, is used. For wind coming from the northeast, the most exposed

of sectors one or two is used to determine full x and y loading individually, and then 75 percent of these loads are to be applied in each direction at the same time according to the requirements of Section 207B.4.6 and Figure 207B.4-8. The procedure defined in this section for determining wind loads in each design direction is not to be confused with the determination of the wind directionality factor K_d . The K_d factor determined from Section 207A.6 and Table 207A.6-1 applies for all design wind directions. See Section C207A.6.

Wind loads for cladding and low-rise buildings elements are determined using the upwind exposure for the single surface roughness in one of the eight sectors of Figure C207A.7-5 that gives the highest cladding pressures.

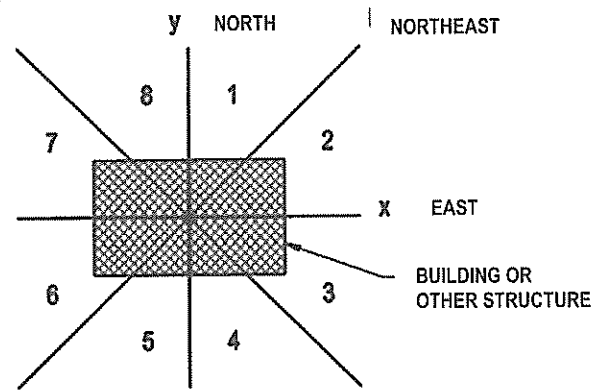
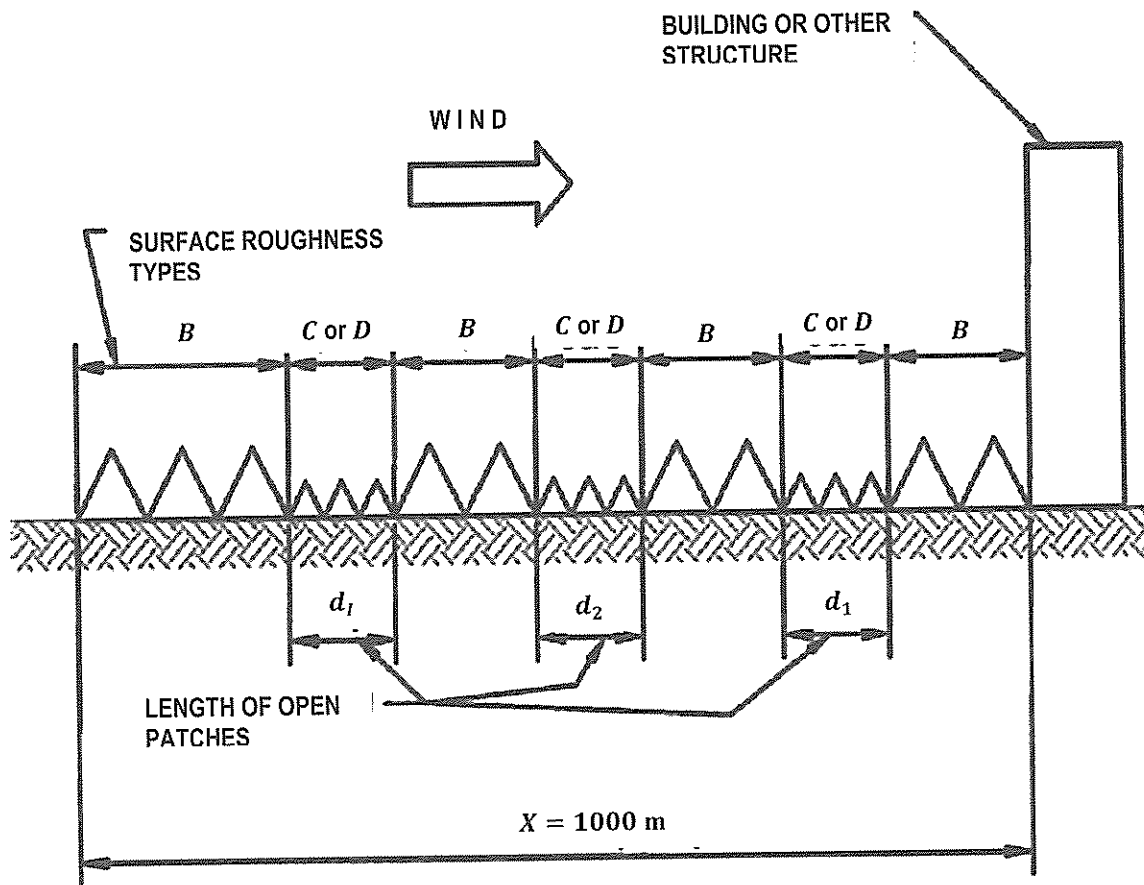


Figure C207A.7-5
Determination of Wind Loads
from Different Directions



"OPEN PATCHES" – OPENINGS $\geq 50 \text{ m} \times 50 \text{ m}$
 $d_1, d_2, \dots, d_i \geq 50 \text{ m}$
 $d_1 + d_2 + \dots + d_i \leq 200 \text{ m}$
 TOTAL LENGTH OF SURFACE ROUGHNESS $B \geq 800 \text{ m}$
 WITHIN 1000 m OF UPWIND FETCH DISTANCE

Figure C207A.7-3
 Exposure B with Upwind Open Patches

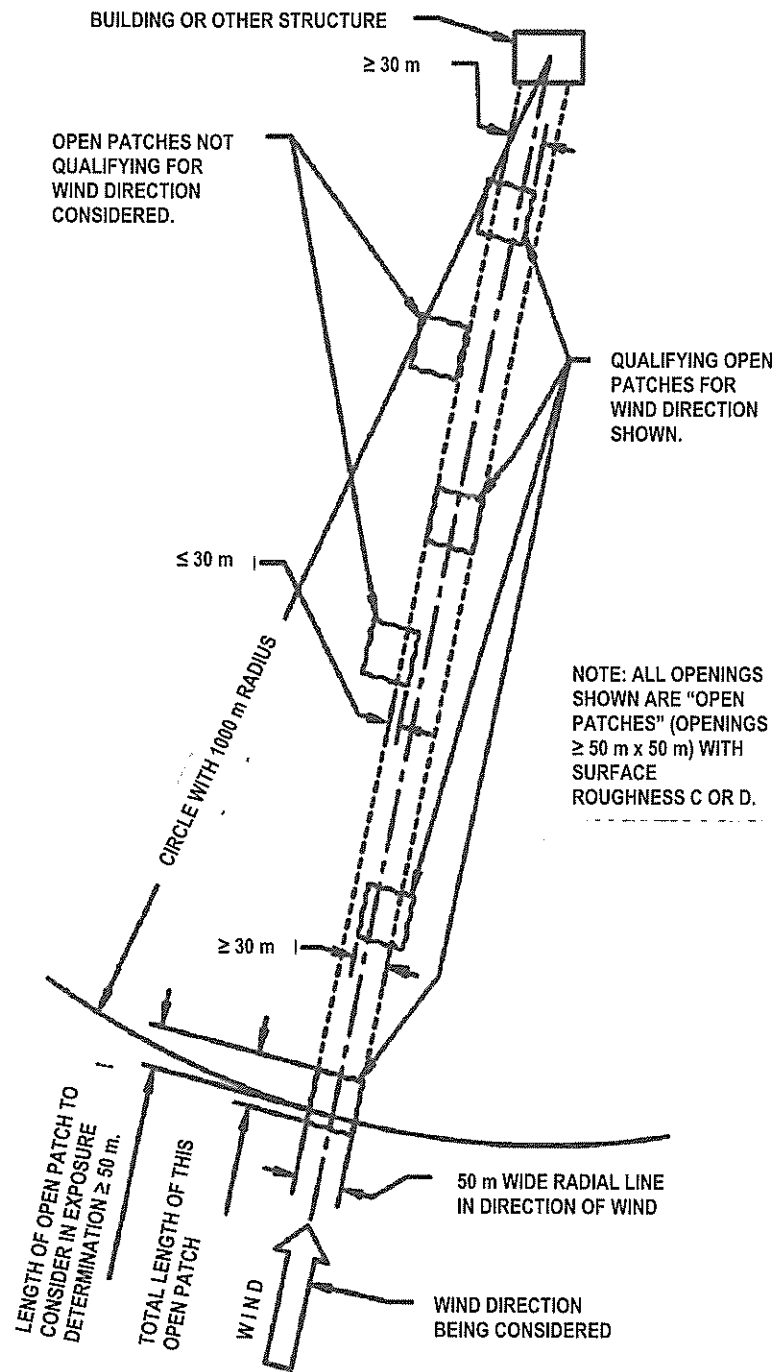


Figure C207A.7-4
Exposure B with Open Patches

207A.7.4.1 Directional Procedure (Section 207B)

For each wind direction considered, wind loads for the design of the MWFRS of enclosed and partially enclosed buildings using the Directional Procedure of Section 207B shall be based on the exposures as defined in Section 207A.7.3. Wind loads for the design of open buildings with monoslope, pitched, or troughed free roofs shall be based on the exposures, as defined in Section 207A.7.3, resulting in the highest wind loads for any wind direction at the site.

207A.7.4.2 Envelope Procedure (Section 207C)

Wind loads for the design of the MWFRS for all low-rise buildings designed using the Envelope Procedure of Section 207C shall be based on the exposure category resulting in the highest wind loads for any wind direction at the site.

207A.7.4.3 Directional Procedure for Building Appurtenances and Other Structures (Section 207D)

Wind loads for the design of building appurtenances (such as rooftop structures and equipment) and other structures (such as solid freestanding walls and freestanding signs, chimneys, tanks, open signs, lattice frameworks, and trussed towers) as specified in Section 207D shall be based on the appropriate exposure for each wind direction considered.

207A.7.4.4 Components and Cladding (Section 207E)

Design wind pressures for components and cladding shall be based on the exposure category resulting in the highest wind loads for any wind direction at the site.

207A.8 Topographic Effects*Commentary:*

As an aid to the designer, this section was rewritten in ASCE 7-98 to specify when topographic effects need to be applied to a particular structure rather than when they do not as in the previous version. In an effort to exclude situations where little or no topographic effect exists, Condition (2) was added to include the fact that the topographic feature should protrude significantly above (by a factor of two or more) upwind terrain features before it becomes a factor. For example, if a significant upwind terrain feature has a height of 10 m above its base elevation and has a top elevation of 30 m above mean sea level then the topographic feature (hill, ridge, or escarpment) must have at least the H specified and extend to elevation 52 m

mean sea level ($30.5 \text{ m} + [3.22 \times 10.7 \text{ m}]$) within the 3.22-km radius specified.

A wind tunnel study by Means et al. (1996) and observation of actual wind damage has shown that the affected height H is less than previously specified. Accordingly, Condition (5) was changed to 4.5 m in Exposure C.

Buildings sited on the upper half of an isolated hill or escarpment may experience significantly higher wind speeds than buildings situated on level ground. To account for these higher wind speeds, the velocity pressure exposure coefficients in Tables 207B.3-1, 207C.3-1, 207D.3-1, and 207E.3-1 are multiplied by a topographic factor, K_{zt} , determined by Equation 207A.8-1. The topographic feature (2-D ridge or escarpment, or 3-D axisymmetrical hill) is described by two parameters, H and L_h . H is the height of the hill or difference in elevation between the crest and that of the upwind terrain. L_h is the distance upwind of the crest to where the ground elevation is equal to half the height of the hill. K_{zt} is determined from three multipliers, K_1 , K_2 , and K_3 , which are obtained from Figure 207A.8-1, respectively. K_1 is related to the shape of the topographic feature and the maximum speed-up near the crest, K_2 accounts for the reduction in speed-up with distance upwind or downwind of the crest, and K_3 accounts for the reduction in speed-up with height above the local ground surface.

The multipliers listed in Figure 207A.8-1 are based on the assumption that the wind approaches the hill along the direction of maximum slope, causing the greatest speed-up near the crest. The average maximum upwind slope of the hill is approximately $H/2L_h$, and measurements have shown that hills with slopes of less than about 0.10 ($H/L_h < 0.20$) are unlikely to produce significant speed-up of the wind. For values of $H/L_h > 0.5$ the speed-up effect is assumed to be independent of slope. The speed-up principally affects the mean wind speed rather than the amplitude of the turbulent fluctuations, and this fact has been accounted for in the values of K_1 , K_2 , and K_3 given in Figure 207A.8-1. Therefore, values of K_{zt} obtained from Figure 207A.8-1 are intended for use with velocity pressure exposure coefficients, K_h and K_z , which are based on gust speeds.

It is not the intent of Section 207A.8 to address the general case of wind flow over hilly or complex terrain for which engineering judgment, expert advice, or the Wind Tunnel Procedure as described in Section 207F may be required. Background material on topographic speed-up effects may be found in the literature (Jackson and Hunt 1975, Lemelin et al. 1988, and Walmsley et al. 1986).

The designer is cautioned that, at present, the code contains no provision for vertical wind speed-up because of a topographic effect, even though this phenomenon is known to exist and can cause additional uplift on roofs. Additional research is required to quantify this effect before it can be incorporated into the code.

207A.8.1 Wind Speed-Up over Hills, Ridges, and Escarpments

Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography, located in any exposure category, shall be included in the design when buildings and other site conditions and locations of structures meet all of the following conditions:

1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature ($100H$) or 3.2 km, whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined.

2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 3.2-km radius in any quadrant by a factor of two or more.
3. The structure is located as shown in Figure 207A.8-1 in the upper one-half of a hill or ridge or near the crest of an escarpment.
4. $H/L_h \geq 0.2$.
5. H is greater than or equal to 4.5 m for Exposure C and D and 18 m for Exposure B.

207A.8.2 Topographic Factor

The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt} :

$$K_{zt} = (1 + K_1 + K_2 + K_3)^2 \quad (207A.8-1)$$

where K_1 , K_2 , and K_3 are given in Figure 207A.8-1.

If site conditions and locations of structures do not meet all the conditions specified in Section 207A.8.1 then $K_{zt} = 1.0$.

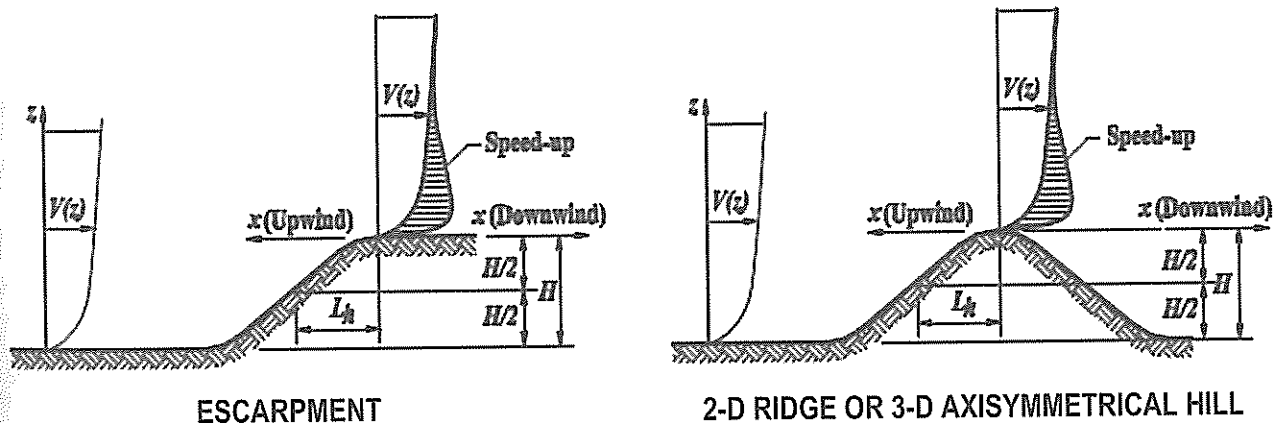


Figure 207A.8-1
Topographic Factor, K_{zt}

Table 207A.8-1
Topographic Multipliers for Exposure C

H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2-D Ridge	2-D Escarp.	3-D Axisym. Hill		2-D Escarp.	All Other Cases		2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

- For values of H/L_h , x/L_h and z/L_h other than those shown, linear interpolation is permitted.
- For $\frac{H}{L_h} > 0.50$, assume $\frac{H}{L_h} = 0.50$ for evaluating K_1 and substitute $2H$ for L_h for evaluating K_2 and K_3 .
- Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
- Notation:

- H = Height of hill or escarpment relative to the upwind terrain, in meters.
 L_h = Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in meters.
 K_1 = Factor to account for shape of topographic feature and maximum speed-up effect.
 K_2 = Factor to account for reduction in speed-up with distance upwind or downwind of crest.
 K_3 = Factor to account for reduction in speed-up with height above local terrain.
 x = Distance (upwind or downwind) from the crest to the building site, in meters.
 z = Height above ground surface at building site, in meters.
 μ = Horizontal attenuation factor.
 γ = Height attenuation factor.

Equations:

$$K_{zt} = (1 + K_1 + K_2 + K_3)^2$$

K_1 determined from table below

$$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$$

$$K_3 = e^{-\gamma z/L_h}$$

Hill Shape	$K_1/(H/L_h)$			γ	μ	
	Exposure				Upwind of Crest	Downwind of Crest
	B	C	D			
2-dimensional ridges (or valleys with negative) H in $K_1/(H/L_h)$	1.30	1.45	1.55	3.00	1.50	1.50
2-dimensional escarpments	0.75	0.85	0.95	2.50	1.50	4.00
3-dimensional axisymmetrical hill	0.95	1.05	1.15	4.00	1.50	1.50

Figure 207A.8-2
Parameters for Speed-Up Over Hills and Escarpments

207A.9 Gust Effects

Commentary:

NSCP 2001 contains a single gust effect factor of 0.85 for rigid buildings. As an option, the designer can incorporate specific features of the wind environment and building size to more accurately calculate a gust effect factor. One such procedure is located in the body of the standard (Solari 1993a and 1993b). A procedure is also included for calculating the gust effect factor for flexible structures. The rigid structure gust factor is 0 percent to 10 percent lower than the simple, but conservative, value of 0.85 permitted in the code without calculation. The procedures for both rigid and flexible structures (1) provide a superior model for flexible structures that displays the peak factors g_Q and g_R and (2) cause the flexible structure value to match the rigid structure as resonance is removed. A designer is free to use any other rational procedure in the approved literature, as stated in Section 207A.9.5.

The gust effect factor accounts for the loading effects in the along-wind direction due to wind turbulence-structure interaction. It also accounts for along-wind loading effects due to dynamic amplification for flexible buildings and structures. It does not include allowances for across-wind loading effects, vortex shedding, instability due to galloping or flutter, or dynamic torsional effects. For structures susceptible to loading effects that are not accounted for in the gust effect factor, information should be obtained from recognized literature (Kareem 1992 and 1985, Gurley and Kareem 1993, Solari 1993a and 1993b, and Kareem and Smith 1994) or from wind tunnel tests.

Along-Wind Response. Based on the preceding definition of the gust effect factor, predictions of along wind response, for example, maximum displacement, root-mean-square (rms), and peak acceleration, can be made. These response components are needed for survivability and serviceability limit states. In the following, expressions for evaluating these along-wind response components are given.

Maximum Along-Wind Displacement. The maximum along-wind displacement $X_{\max}(z)$ as a function of height above the ground surface is given by:

$$X_{\max}(z) = \frac{\phi(z)\rho B h C_{fx} \bar{V}_z^2}{2m_1(2\pi n_1)^2} K G \quad (C207A.9-1)$$

where

$$\begin{aligned} \phi(z) &= \text{the fundamental model shape} \\ &= (z/h)^\xi \\ \xi &= \text{the mode exponent} \\ \rho &= \text{air density} \\ C_{fx} &= \text{mean along-wind force coefficient} \\ m_1 &= \text{modal mass} \\ &= \int_0^h \mu(z) \phi^2(z) dz \\ \mu(z) &= \text{mass per unit height} \\ K &= (1.65)^\alpha / (\hat{\alpha} + \xi + 1) \end{aligned}$$

and $\bar{V}_z$ is the 3-s gust speed at height z . This can be evaluated by $\bar{V}_z = \bar{b}(z/33)^{\hat{\alpha}} V$, where V is the 3-s gust speed in Exposure C at the reference height (obtained from Figure 207A.5-1); $\bar{b}$ and $\hat{\alpha}$ are given in Table 207A.9-1.

RMS Along-Wind Acceleration. The rms along-wind acceleration $\sigma_{\ddot{x}}(z)$ as a function of height above the ground surface is given by:

$$\sigma_{\ddot{x}}(z) = \frac{0.85\phi(z)\rho B h C_{fx} \bar{V}_z^2}{m_1} I_z K R \quad (C207A.9-2)$$

where $\bar{V}_z$ is the mean hourly wind speed at height z , m/s:

$$\bar{V}_z = \bar{b} \left(\frac{z}{33} \right)^{\hat{\alpha}} V \quad (C207A.9-3)$$

where $\bar{b}$ and $\hat{\alpha}$ are defined in Table 207A.9-1.

Maximum Along-Wind Acceleration. The maximum along-wind acceleration as a function of height above the ground surface is given by:

$$\ddot{X}_{\max}(z) = g_{\ddot{x}} \sigma_{\ddot{x}}(z) \quad (C207A.9-4)$$

$$g_{\ddot{x}} = \sqrt{2 \ln(n_1 T)} + \frac{0.5772}{\sqrt{2 \ln(n_1 T)}} \quad (C207A.9-5)$$

where T = the length of time over which the minimum acceleration is computed, usually taken to be 3,600 s to represent 1 hour.

Approximate Fundamental Frequency. To estimate the dynamic response of structures, knowledge of the fundamental frequency (lowest natural frequency) of the structure is essential. This value would also assist in determining if the dynamic response estimates are necessary. Most computer codes used in the analysis of

structures would provide estimates of the natural frequencies of the structure being analyzed. However, for the preliminary design stages some empirical relationships for building period T_a ($T_a = 1/n_1$) are available in the earthquake chapters of ASCE 7. However, it is noteworthy that these expressions are based on recommendations for earthquake design with inherent bias toward higher estimates of fundamental frequencies (Goel and Chopra 1997 and 1998). For wind design applications these values may be unconservative because an estimated frequency higher than the actual frequency would yield lower values of the gust effect factor and concomitantly a lower design wind pressure. However, Goel and Chopra (1997 and 1998) also cite lower bound estimates of frequency that are more suited for use in wind applications. These lower-bound expressions are now given in Section 207A.9.2; graphs of these expressions are shown in Figure C207A.9-1. Because these expressions are based on regular buildings, limitations based on height and slenderness are required. The effective length L_{eff} uses a height-weighted average of the along-wind length of the building for slenderness evaluation. The top portion

of the building is most important; hence the height-weighted average is appropriate. This method is an appropriate first-order equation for addressing buildings with setbacks. Explicit calculation of gust effect factor per the other methods given in Section 207A.9 can still be performed.

Observation from wind tunnel testing of buildings where frequency is calculated using analysis software reveals the following expression for frequency, appropriate for buildings less than about 120 m in height, applicable to all buildings in steel or concrete:

$$n_1 = 100/H \text{ (m) average value} \quad (C207A.9-6)$$

$$n_1 = 75/H \text{ (m) lower bound value} \quad (C207A.9-7)$$

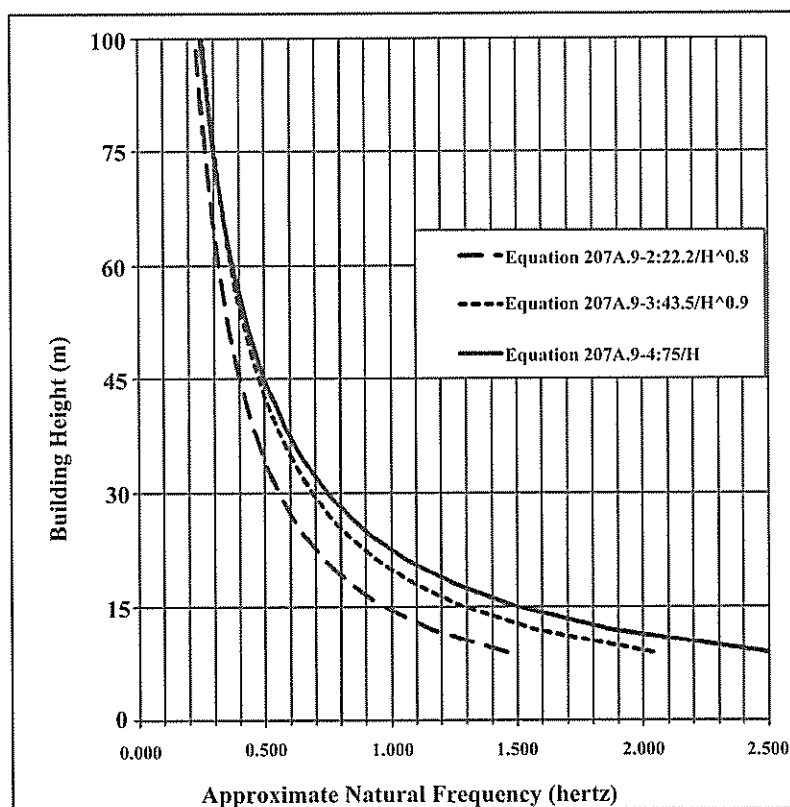


Figure C207A.9-1
Equations for Approximate Natural Frequency vs. Building Height

Equation C207A.9-7 for the lower bound value is provided in Section 207A.9.2.

Based on full-scale measurements of buildings under the action of wind, the following expression has been proposed for wind applications (Zhou and Kareem 2001, Zhou, Kijewski, and Kareem 2002):

$$f_{n1} = 150/H \quad (\text{C207A.9-8})$$

This frequency expression is based on older buildings and overestimates the frequency common in U.S. construction for smaller buildings less than 120 m in height, but becomes more accurate for tall buildings greater than 120 m in height. The Australian and New Zealand Standard AS/NZS 1170.2, Eurocode ENV1991-2-4, Hong Kong Code of Practice on Wind Effects (2004), and others have adopted Equation C207A.9-8 for all building types and all heights.

Recent studies in Japan involving a suite of buildings under low-amplitude excitations have led to the following expressions for natural frequencies of buildings (Sataka et al. 2003):

$$n_1 = 220/H(m) \text{ (concrete buildings)} \quad (\text{C207A.9-9})$$

$$n_1 = 164/H(m) \text{ (steel buildings)} \quad (\text{C207A.9-10})$$

The expressions based on Japanese buildings result in higher frequency estimates than those obtained from the general expression given in Equations C207A.9-6 through C207A.9-8, particularly since the Japanese data set has limited observations for the more flexible buildings sensitive to wind effects and Japanese construction tends to be stiffer.

For cantilevered masts or poles of uniform cross-section (in which bending action dominates):

$$n_1 = (0.56/h^2)\sqrt{(EI/m)} \quad (\text{C207A.9-11})$$

where EI is the bending stiffness of the section and m is the mass/unit height. (This formula may be used for masts with a slight taper, using average value of EI and m) (ECCS 1978).

An approximate formula for cantilevered, tapered, circular poles (ECCS 1978) is:

$$n_1 = [\lambda/(2\pi h^2)]\sqrt{(EI/m)} \quad (\text{C207A.9-12})$$

where h is the height, and E , I , and m are calculated for the cross-section at the base. λ depends on the wall thicknesses at the tip and base, e_t and e_b , and external diameter at the tip and base, d_t and d_b , according to the following formula:

$$\lambda = \left[1.9 \exp\left(\frac{-4d_t}{d_b}\right) \right] + \left[\frac{6.65}{0.9 + \left(\frac{e_t}{e_b}\right)^{0.666}} \right] \quad (\text{C207A.9-13})$$

Equation C207A.9-12 reduces to Equation C207A.9-11 for uniform masts. For free-standing lattice towers (without added ancillaries such as antennas or lighting frames) (Standards Australia 1994):

$$n_1 = 1500w_a/h^2 \quad (\text{C207A.9-14})$$

where w_a is the average width of the structure in m and h is tower height. An alternative formula for lattice towers (with added ancillaries) (Wyatt 1984) is:

$$n_1 = \left(\frac{L_N}{H}\right)^{2/3} \left(\frac{w_b}{H}\right)^{1/2} \quad (\text{C207A.9-15})$$

where w_b = tower base width and L_N = 270 m for square base towers, or 230 m for triangular base towers.

Structural Damping. Structural damping is a measure of energy dissipation in a vibrating structure that results in bringing the structure to a quiescent state. The damping is defined as the ratio of the energy dissipated in one oscillation cycle to the maximum amount of energy in the structure in that cycle. There are as many structural damping mechanisms as there are modes of converting mechanical energy into heat. The most important mechanisms are material damping and interfacial damping.

In engineering practice, the damping mechanism is often approximated as viscous damping because it leads to a linear equation of motion. This damping measure, in terms of the damping ratio, is usually assigned based on the construction material, for example, steel or concrete. The calculation of dynamic load effects requires damping ratio as an input. In wind applications, damping ratios of 1 percent and 2 percent are typically used in the United States for steel and concrete buildings at serviceability levels, respectively,

while ISO (1997) suggests 1 percent and 1.5 percent for steel and concrete, respectively. Damping values for steel support structures for signs, chimneys, and towers may be much lower than buildings and may fall in the range of 0.15 percent to 0.5 percent. Damping values of special structures like steel stacks can be as low as 0.2 percent to 0.6 percent and 0.3 percent to 1.0 percent for unlined and lined steel chimneys, respectively (ASME 1992 and CICIND 1999). These values may provide some guidance for design. Damping levels used in wind load applications are smaller than the 5 percent damping ratios common in seismic applications because buildings subjected to wind loads respond essentially elastically whereas buildings subjected to design level earthquakes respond inelastically at higher damping levels.

Because the level of structural response in the serviceability and survivability states is different, the damping values associated with these states may differ. Further, due to the number of mechanisms responsible for damping, the limited full-scale data manifest a dependence on factors such as material, height, and type of structural system and foundation. The Committee on Damping of the Architectural Institute of Japan suggests different damping values for these states based on a large damping database described in Sataka et al. (2003).

In addition to structural damping, aerodynamic damping may be experienced by a structure oscillating in air. In general, the aerodynamic damping contribution is quite small compared to the structural damping, and it is positive in low to moderate wind speeds. Depending on the structural shape, at some wind velocities, the aerodynamic damping may become negative, which can lead to unstable oscillations. In these cases, reference should be made to recognized literature or a wind tunnel study.

Alternate Procedure to Calculate Wind Loads. The concept of the gust effect factor implies that the effect of gusts can be adequately accounted for by multiplying the mean wind load distribution with height by a single factor. This is an approximation. If a more accurate representation of gust effects is required, the alternative procedure in this section can be used. It takes account of the fact that the inertial forces created by the building's mass, as it moves under wind action, have a different distribution with height than the mean wind loads or the loads due to the direct actions of gusts (ISO 1997 and Sataka et al. 2003). The alternate formulation of the equivalent static load distribution utilizes the peak base bending moment and expresses it in terms of inertial forces at different building levels. A base bending moment, instead of the base shear as in

earthquake engineering, is used for the wind loads, as it is less sensitive to deviations from a linear mode shape while still providing a gust effect factor generally equal to the gust factor calculated by the Section 207 NSCP 2010 standard. This equivalence occurs only for structures with linear mode shape and uniform mass distribution, assumptions tacitly implied in the previous formulation of the gust effect factor, and thereby permits a smooth transition from the existing procedure to the formulation suggested here. For a more detailed discussion on this wind loading procedure, see ISO (1997) and Sataka et al. (2003).

Along-Wind Equivalent Static Wind Loading. The equivalent static wind loading for the mean, background, and resonant components is obtained using the procedure outlined in the following text.

Mean wind load component $\bar{P}_j$ at the j^{th} floor level is given by:

$$\bar{P}_j = q_j \times C_p \times A_j \times \bar{G} \quad (\text{C207A.9-16})$$

where

- j = floor level
- z_j = height of the j^{th} floor above the ground level
- q_j = velocity pressure at height z_j
- C_p = external pressure coefficient
- G = $0.925(1 + 1.7g_v I_z)^{-1}$ is the gust velocity factor

Peak background wind load component $\hat{P}_{Bj}$ at the j^{th} floor level is given similarly by:

$$\hat{P}_{Bj} = \bar{P}_j \left(\frac{G_B}{G} \right) \quad (\text{C207A.9-17})$$

where

$$\begin{aligned} G_B &= 0.925 \left(\frac{1.7I_z g_Q Q}{1 + 1.7g_v I_z} \right) \\ &= \text{is the background component of the gust effect factor.} \end{aligned}$$

Peak resonant wind load component $\hat{P}_{Rj}$ at the j^{th} floor level is obtained by distributing the resonant base bending moment response to each level

$$\hat{P}_{Rj} = C_{Mj} \hat{M}_R \quad (\text{C207A.9-18})$$

$$C_{Mj} = \frac{w_j \phi_j}{\sum w_j \phi_j z_j} \quad (C207A.9-19)$$

$$\bar{M}_R = \frac{\bar{M} G_R}{G} \quad (C207A.9-20)$$

$$\bar{M} = \sum_{j=1,n} \bar{P}_j z_j \quad (C207A.9-21)$$

where

- C_{Mj} = vertical load distribution factor
 $\bar{M}_R$ = peak resonant component of the base bending moment response
 w_j = portion of the total gravity load of the building located or assigned to level j
 n = total stories of the building
 ϕ_j = first structural mode shape value at level j
 $\bar{M}$ = mean base bending produced by mean wind load

$$G_R = 0.925 \left(\frac{1.7 I_z G_R R}{1 + 1.7 G_v I_z} \right)$$

= is the resonant component of the gust effect factor.

Along-Wind Response. Through a simple static analysis the peak-building response along-wind direction can be obtained by:

$$\hat{r} = \bar{r} + \sqrt{\hat{r}_B^2 + \hat{r}_R^2} \quad (C207A.9-22)$$

where $\bar{r}$, $\hat{r}_B$, and $\hat{r}_R$ = mean, peak background, and resonant response components of interest, for example, shear forces, moment, or displacement. Once the equivalent static wind load distribution is obtained, any response component including acceleration can be obtained using a simple static analysis. It is suggested that caution must be exercised when combining the loads instead of response according to the preceding expression, for example,

$$\bar{P}_j = \bar{P}_j + \sqrt{\bar{P}_{Bj}^2 + \bar{P}_{Rj}^2} \quad (C207A.9-23)$$

because the background and the resonant load components have normally different distributions along

the building height. Additional background can be found in ISO (1997) and Sataka et al. (2003).

Example: The following example is presented to illustrate the calculation of the gust effect factor. Table C207A.9-1 uses the given information to obtain values from Table 207A.9-1. Table C207A.9-2 presents the calculated values. Table C207A.9-3 summarizes the calculated displacements and accelerations as a function of the height, z .

Given Values:

Basic wind speed at reference height in exposure C	= 150 km/h
C_{fx}	= 1.3
Damping ratio	= 0.01
Mode exponent	= 1.0
Type of exposure	= B
Building height h	= 183 m
Building width B	= 30.5 m
Building depth L	= 30.5 m
Building natural frequency n_1	= 0.2 Hz
Building density	= 12 lb/ft ³ 192.0817 kg/m ³
Air density	= 1.2369 kg/m ³

Table C207A.9-1

Calculated Values

$z_{min}(m)^*$	= 9.14 ft
$\bar{e}$	= 1/3
c	= 0.3
$\bar{b}$	= 0.45
$\bar{a}$	= 0.25
$\hat{b}$	= 0.84
$\hat{a}$	= 1/7
l	= 97.54 m
ξ	= 1

Table C207A.9-2
Calculated Values

V	40.23 m/s
$\bar{z}$	109.73 m
$I_{\bar{z}}$	0.201
$L_{\bar{z}}$	216.75 m
Q^2	0.616
$\bar{V}_{\bar{z}}$	32.95 m/s
$\hat{V}_{\bar{z}}$	47.59 m/s
N_1	1.31
R_n	0.113
η	0.852

R_B	0.610
η	5.113
R_h	0.176
η	2.853
R_L	0.289
R^2	0.813
G_f	1.062
K	0.501
m_1	10.88x10 ⁶ kg
g_R	3.787

Table 207A.9-3
Along-wind Response - Example

Floor	z_j (m)	ϕ_j	$X_{max j}$	RMS Acc.* (m/s ²)	RMS Acc.* (mg)	Max Acc.* (m/s ²)	Max Acc.* (mg)
0	0	0	0	0	0	0	0
5	18.29	0.10	0.03	0.00	0.41	0.02	1.6
10	36.58	0.20	0.06	0.01	0.83	0.03	3.1
15	54.86	0.30	0.09	0.01	1.24	0.05	4.7
20	73.15	0.40	0.13	0.02	1.66	0.06	6.3
25	91.44	0.50	0.16	0.02	2.07	0.08	7.8
30	109.73	0.60	0.19	0.02	2.49	0.09	9.4
35	128.02	0.70	0.22	0.03	2.90	0.11	11.0
40	146.3	0.80	0.25	0.03	3.32	0.12	12.6
45	164.59	0.80	0.28	0.04	3.73	0.14	14.1
50	182.88	1.00	0.31	0.04	4.14	0.15	15.7

Aerodynamic Loads on Tall Buildings—An Interactive Database. Under the action of wind, tall buildings oscillate simultaneously in the along-wind, across-wind, and torsional directions. While the along-wind loads have been successfully treated in terms of gust loading factors based on quasi-steady and strip theories, the across-wind and torsional loads cannot be treated in this manner, as these loads cannot be related in a straightforward manner to fluctuations in the approach flow. As a result, most current codes and standards provide little guidance for the across-wind and torsional response ISO (1997) and Satoka et al. (2003).

To provide some guidance at the preliminary design stages of buildings, an interactive aerodynamic loads database for assessing dynamic wind-induced loads on a suite of generic isolated buildings is introduced. Although the analysis based on this experimental database is not intended to replace wind tunnel testing in the final design stages, it provides users a methodology to approximate the previously untreated

across-wind and torsional responses in the early design stages. The database consists of high frequency base balance measurements involving seven rectangular building models, with side ratio D/B , where D is the depth of the building section along the oncoming wind direction) from 1/3 to 3, three aspect ratios for each building model in two approach flows, namely, $BL_1(\bar{\alpha} = 0.16)$ and $BL_2(\bar{\alpha} = 0.35)$ corresponding to an open and an urban environment. The data are accessible with a user-friendly Java-based applet through the worldwide Internet community at <http://aerodata.ce.nd.edu/interface/interface.html>.

Through the use of this interactive portal, users can select the geometry and dimensions of a model building from the available choices and specify an urban or suburban condition. Upon doing so, the aerodynamic load spectra for the along-wind, across-wind, or torsional directions is displayed with a Java interface permitting users to specify a reduced frequency (building frequency $\times$ building dimension/ wind velocity) of interest and automatically obtain the

corresponding spectral value. When coupled with the supporting Web documentation, examples, and concise analysis procedure, the database provides a comprehensive tool for computation of wind-induced response of tall buildings, suitable as a design guide in the preliminary stages.

Example: An example tall building is used to demonstrate the analysis using the database. The building is a square steel tall building with size $H \times W_1 \times W_2 = 200 \times 40 \times 40$ m and an average radius of gyration of 18 m.

The three fundamental mode frequencies, f_1 , are 0.2, 0.2, and 0.35 Hz in X, Y, and Z directions, respectively; the mode shapes are all linear, or β is equal to 1.0, and there is no modal coupling. The building density is equal to 250 kg/m³. This building is located in Exposure A or close to the **BL₂** test condition of the Internet-based database (Zhou et al. 2002). In this location (Exposure A), the reference 3-sec design gust speed at a 50-year recurrence interval is 63 m/s [ASCE 7-98], which is equal to 18.9 m/s upon conversion to 1-h mean wind speed with 50-yr MRI ($207 \times 0.30 = 62$ m/s). For serviceability requirements, 1-h mean wind speed with 10-yr MRI is equal to 14 m/s ($207 \times 0.30 \times 0.74 = 46$). For the sake of illustration only, the first mode critical structural damping ratio, ζ_1 , is to be 0.01 for both survivability and serviceability design.

Using these aerodynamic data and the procedures provided on the Web and in ISO (1997), the wind load effects are evaluated and the results are presented in Table C207A.9-4. This table includes base moments and acceleration response in the along-wind direction obtained by the procedure in ASCE 7-02. Also the building experiences much higher across-wind load effects when compared to the along-wind response for this example, which reiterates the significance of wind loads and their effects in the across-wind direction.

207A.9.1 Gust Effect Factor

The gust-effect factor for a rigid building or other structure is permitted to be taken as 0.85.

207A.9.2 Frequency Determination

To determine whether a building or structure is rigid or flexible as defined in Section 207A.2, the fundamental natural frequency, n_1 , shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis.

Low-Rise Buildings, as defined in 207A.2, are permitted to be considered rigid.

207A.9.2.1 Limitations for Approximate Natural Frequency

As an alternative to performing an analysis to determine n_1 , the approximate building natural frequency, n_1 , shall be permitted to be calculated in accordance with Section 207A.9.3 for structural steel, concrete, or masonry buildings meeting the following requirements:

1. The building height is less than or equal to 91 m, and
2. The building height is less than 4 times its effective length, L_{eff} .

The effective length, L_{eff} , in the direction under consideration shall be determined from the following equation:

$$L_{eff} = \frac{\sum_{i=1}^n h_i L_i}{\sum_{i=1}^n h_i} \quad (207A.9-1)$$

The summations are over the height of the building

where

- h_i = is the height above grade of level i
 L_i = is the building length at level i parallel to the wind direction

207A.9.3 Approximate Natural Frequency

The approximate lower-bound natural frequency (n_a), in Hertz, of concrete or structural steel buildings meeting the conditions of Section 207A.9.2.1, is permitted to be determined from one of the following equations:

For structural steel moment-resisting-frame buildings:

$$n_a = 22.2/h^{0.8} \quad (207A.9-2)$$

For concrete moment-resisting frame buildings:

$$n_a = 43.5/h^{0.9} \quad (207A.9-3)$$

For structural steel and concrete buildings with other lateral-force-resisting systems:

$$n_a = 75/h \quad (207A.9-4)$$

For concrete or masonry shear wall buildings, it is also permitted to use:

$$n_a = 385(C_w)^{0.5}/h \quad (207A.9-5)$$

where

$$C_w = \frac{100}{A_B} \sum_{i=1}^n \left(\frac{h}{h_i} \right)^2 \frac{A_i}{\left[1 + 0.83 \left(\frac{h_i}{D_i} \right)^2 \right]}$$

where

- h = mean roof height (m)
- n = number of shear walls in the building effective in resisting lateral forces in the direction under consideration
- A_B = base area of the structure (m²)
- A_i = horizontal cross-section area of shear wall "i" (m²)
- D_i = length of shear wall "i" (m)
- h_i = height of shear wall "i" (m)

207A.9.4 Rigid Buildings or Other Structures

For rigid buildings or other structures as defined in Section 207A.2, the gust-effect factor shall be taken as 0.85 or calculated by the formula:

$$G = 0.925 \left(\frac{1 + 1.7g_Q I_z Q}{1 + 1.7g_v I_z} \right) \quad (207A.9-6)$$

$$I_z = c \left(\frac{33}{\bar{z}} \right)^{1/6} \quad (207A.9-7)$$

In SI:

$$I_z = c \left(\frac{10}{\bar{z}} \right)^{1/6}$$

where I_z is the intensity of turbulence at height $\bar{z}$

where $\bar{z}$ is the equivalent height of the structure defined as $0.6h$, but not less than z_{min} for all building heights h . z_{min} and c are listed for each exposure in Table 207A.9-1; g_Q and g_v shall be taken as 3.4. The background response Q is given by:

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{B+h}{L_z} \right)^{0.63}}} \quad (207A.9-8)$$

where B and h are defined in Section 207A.3 and L_z is the integral length scale of turbulence at the equivalent height given by:

$$L_z = \ell \left(\frac{\bar{z}}{33} \right)^{\bar{\epsilon}} \quad (207A.9-9)$$

In SI:

$$L_z = \ell \left(\frac{\bar{z}}{10} \right)^{\bar{\epsilon}}$$

In which ℓ and $\bar{\epsilon}$ are constants listed in Table 207A.9-1.

207A.9.5 Flexible or Dynamically Sensitive Buildings or Other Structures

For flexible or dynamically sensitive buildings or other structures as defined in Section 207A.2, the gust-effect factor shall be calculated by:

$$G_f = \frac{1 + 1.7I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7g_v I_z} \quad (207A.9-10)$$

g_Q and g_v shall be taken as 3.4 and g_R is given by:

$$g_R = \sqrt{\frac{2 \ln(3600n_1)}{0.577} + \frac{0.577}{2 \ln(3600n_1)}} \quad (207A.9-11)$$

R , the resonant response factor, is given by:

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \quad (207A.9-12)$$

$$R_n = \frac{7.47 N_i}{(1 + 10.3 N_i)^{5/3}} \quad (207A.9-13)$$

$$N_1 = \frac{n_1 L_z}{V_z} \quad (207A.9-14)$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2} (1 - e^{-2\eta}) \quad \text{for } \eta > 0 \quad (207A.9-15a)$$

$$R_\ell = 1 \quad \text{for } \eta = 0 \quad (207A.9-15b)$$

where the subscript ℓ in Equation 207A.9-15 shall be taken as h , B , and L , respectively, where h , B , and L are defined in Section 207A.3.

- n_1 = fundamental natural frequency
- R_ℓ = R_h setting $\eta = 4.6n_1h/\bar{V}_z$
- R_ℓ = R_B setting $\eta = 4.6n_1B/\bar{V}_z$
- R_ℓ = R_L setting $\eta = 15.4n_1L/\bar{V}_z$
- β = damping ratio, percent of critical (i.e. for 2% use 0.02 in the equation)
- $\bar{V}_z$ = mean hourly wind speed (m/s) at height $\bar{z}$ determined from Equation 207A.9-16:

$$\bar{V}_z = \bar{b} \left(\frac{\bar{z}}{33} \right)^{\bar{\alpha}} \left(\frac{88}{60} \right) V \quad (207A.9-16)$$

In SI:

$$\bar{V}_z = \bar{b} \left(\frac{\bar{z}}{10} \right)^{\bar{\alpha}} V$$

where $\bar{b}$ and $\bar{\alpha}$ are constants listed in Table 207A.9-1 and V is the basic wind speed in km/h.

207A.9.6 Rational Analysis

In lieu of the procedure defined in Sections 207A.9.3 and 207A.9.4, determination of the gust-effect factor by any rational analysis defined in the recognized literature is permitted.

207A.9.7 Limitations

Where combined gust-effect factors and pressure coefficients (GC_p), (GC_{pi}), and (GC_{pf}) are given in figures and tables, the gust-effect factor shall not be determined separately.

207A.10 Enclosure Classification

Commentary:

Accordingly, the code requires that a determination be made of the amount of openings in the envelope to assess enclosure classification (enclosed, partially enclosed, or open). "Openings" are specifically defined in this version of the code as "apertures or holes in the building envelope which allow air to flow through the building envelope and which are designed as "open" during design winds." Examples include doors, operable windows, air intake exhausts for air conditioning and or ventilation systems, gaps around doors, deliberate gaps in cladding, and flexible and operable louvers. Once the enclosure classification is known, the designer enters Table 207A.11-1 to select the appropriate internal pressure coefficient.

This version of the code has four terms applicable to enclosure: wind-borne debris regions, glazing, impact-resistant glazing, and impact protective system. "Wind-borne debris regions" are specified to alert the designer to areas requiring consideration of missile impact design and potential openings in the building envelope. "Glazing" is defined as "any glass or transparent or translucent plastic sheet used in windows, doors, skylights, or curtain walls." "Impact resistant glazing" is specifically defined as "glazing that has been shown by testing to withstand the impact of test missiles." "Impact protective systems" over glazing can be shutters or screens designed to withstand wind-borne debris impact. Impact resistance of glazing and protective systems can be tested using the test method specified in ASTM E1886-2005 (2005), with missiles, impact speeds, and pass/fail criteria specified in ASTM E1996-2009 (2009). Other approved test methods are acceptable. Origins of missile impact provisions contained in these standards are summarized in Minor (1994) and Twisdale et al. (1996).

Attention is drawn to Section 207A.10.3, which requires glazing in Category I, II, III, and IV buildings in wind-borne debris regions to be protected with an impact protective system or to be made of impact resistant glazing. The option of unprotected glazing was eliminated for most buildings in the 2005 edition of the standard to reduce the amount of wind and water damage to buildings during design wind storm events.

Table 207A.9-1
Terrain Exposure Constants

Exposure	α	z_g	$\hat{a}$	$\hat{b}$	$\bar{a}$	$\bar{b}$	c	$\ell(m)$	$\bar{e}$	$z_{min}(m)^*$
B	7.0	365.76	1/7	0.84	1/4.0	0.45	0.30	97.54	1/3.0	9.14
C	9.5	274.32	1/9.5	1.00	1/6.5	0.65	0.20	152.4	1/5.0	4.57
D	11.5	213.36	1/11.5	1.07	1/9.0	0.80	0.15	198.12	1/8.0	2.13

* z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is greater of $0.6h$ or z_{min} . For buildings with $h \leq z_{min}$, $\bar{z}$ shall be taken as z_{min} .

Prior to the 2002 edition of the standard, glazing in the lower 18 m of Category II, III, or IV buildings sited in wind-borne debris regions was required to be protected with an impact protective system, or to be made of impact-resistant glazing, or the area of the glazing was assumed to be open. Recognizing that glazing higher than 18 m above grade may be broken by wind-borne debris when a debris source is present, a new provision was added in 2002. With that new provision, aggregate surfaced roofs on buildings within 450 m of the new building need to be evaluated. For example, roof aggregate, including gravel or stone used as ballast that is not protected by a sufficiently high parapet should be considered as a debris source. Accordingly, the glazing in the new building, from 9 m above the source building to grade would need to be protected with an impact protective system or be made of impact-resistant glazing. If loose roof aggregate is proposed for the new building, it too should be considered as a debris source because aggregate can be blown off the roof and be propelled into glazing on the leeward side of the building. Although other types of wind-borne debris can impact glazing higher than 18 m above grade, at these higher elevations, loose roof aggregate has been the predominate debris source in previous wind events. The requirement for protection 9 m above the debris source is to account for debris that can be lifted during flight. The following references provide further information regarding debris damage to glazing: Beason et al. (1984), Minor (1985 and 1994), Kareem (1986), and Behr and Minor (1994).

Although wind-borne debris can occur in just about any condition, the level of risk in comparison to the postulated debris regions and impact criteria may also be lower than that determined for the purpose of standardization. For example, individual buildings may be sited away from likely debris sources that would generate significant risk of impacts similar in magnitude to pea gravel (i.e., as simulated by 2 gram steel balls in impact tests) or butt-on 2 × 4 impacts as required in impact testing criteria. This situation describes a condition of low vulnerability only as a result of limited debris sources within the vicinity of the building. In

other cases, potential sources of debris may be present, but extenuating conditions can lower the risk. These extenuating conditions include the type of materials and surrounding construction, the level of protection offered by surrounding exposure conditions, and the design wind speed. Therefore, the risk of impact may differ from those postulated as a result of the conditions specifically enumerated in the code and the referenced impact standards. The committee recognizes that there are vastly differing opinions, even within the standards committee, regarding the significance of these parameters that are not fully considered in developing standardized debris regions or referenced impact criteria.

Recognizing that the definition of the wind-borne debris regions given in NSCP 2001 (ASCE 7-98) through NSCP 2010 (ASCE 7-05) was largely based on engineering judgment rather than a risk and reliability analysis, the definition of the wind-borne debris regions in ASCE 7-10 for Occupancy Category III and IV buildings and structures has been chosen such that the coastal areas included in the wind-borne debris regions defined with the new wind speed maps are approximately consistent with those given in the prior editions for this risk category. Thus, the new wind speed contours that define the wind-borne debris regions in Section 207A.10.3.1 are not direct conversions of the wind speed contours that are defined in NSCP 2010 (ASCE 7-05) as shown in Table C207A.5-6. As a result of this shift, adjustments are needed to the Wind Zone designations in ASTM E 1996 for the determination of the appropriate missile size for the impact test because the Wind Zones are based on the NSCP 2010 (ASCE 7-05) wind speed maps. Chapter 6.2.2 of ASTM E 1996 should be as follows:

6.2.2 Unless otherwise specified, select the wind zone based on the basic wind speed as follows:

6.2.2.1 Wind Zone 1 – $210 \text{ kph} \leq \text{basic wind speed} < 225 \text{ kph}$.

6.2.2.2 Wind Zone 2 – $225 \text{ kph} \leq \text{basic wind speed} < 240 \text{ kph}$ at greater than 1.6 km from the coastline. The coastline shall be measured from the mean high water mark.

6.2.2.3 Wind Zone 3 – basic wind speed $\geq 240 \text{ kph}$, or basic wind speed $\geq 225 \text{ kph}$ and within 1.6 km of the coastline. The coastline shall be measured from the mean high water mark.

However, While the coastal areas included in the wind-borne debris regions defined in the new wind speed maps for Risk Category II are approximately consistent with those given in NSCP 2010 (ASCE 7-05), significant reductions in the wind-borne debris regions for this risk category occur in the area around Jacksonville, Florida, in the Florida Panhandle, and inland from the coast of North Carolina.

The introduction of separate risk-based maps for different risk categories provides a means for achieving a more risk-consistent approach for defining wind-borne debris regions. The approach selected was to link the geographical definition of the wind-borne debris regions to the wind speed contours in the maps that correspond to the particular risk category. The resulting expansion of the wind-borne debris region for Occupancy Category I and II buildings and structures (wind-borne debris regions in Figure 207A.5-1C that are not part of the wind-borne debris regions defined in Figure 207A.5-1B) was considered appropriate for the types of buildings included in Occupancy Category I and II. A review of the types of buildings and structures currently included in Occupancy Category III suggests that life safety issues would be most important, in the expanded wind-borne debris region, for health care facilities. Consequently, the committee chose to apply the expanded wind-borne debris protection requirement to this type of Occupancy Category III facilities and not to all Occupancy Category III buildings and structures.

207A.10.1 General

For the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open as defined in Section 207A.2.

207A.10.2 Openings

A determination shall be made of the amount of openings in the building envelope for use in determining the enclosure classification.

207A.10.3 Protection of Glazed Openings

Glazed openings in Occupancy Category I, II, III or IV buildings located in tropical cyclone-prone regions shall be protected as specified in this Section.

207A.10.3.1 Wind-borne Debris Regions

Glazed openings shall be protected in accordance with Section 207A.10.3.2 in the following locations:

1. Within 1.6 km of the coastal mean high water line where the basic wind speed is equal to or greater than 58 m/s, or
2. In areas where the basic wind speed is equal to or greater than 63 m/s.

For Occupancy Category III and IV buildings and structures, except health care facilities, the wind-borne debris region shall be based on Figure 207A.5-1A. For Occupancy Category III health care facilities and Occupancy Category II buildings and structures, the wind-borne debris region shall be based on Figure 207A.5-1B. Occupancy Categories shall be determined in accordance with Section 103.

Exception:

Glazing located over 18 m above the ground and over 9 m above aggregate-surfaced roofs, including roofs with gravel or stone ballast, located within 450 m of the building shall be permitted to be unprotected.

207A.10.3.2 Protection Requirements for Glazed Openings

Glazing in buildings requiring protection shall be protected with an impact-protective system or shall be impact-resistant glazing.

Impact-protective systems and impact-resistant glazing shall be subjected to missile test and cyclic pressure differential tests in accordance with ASTM E1996 as applicable. Testing to demonstrate compliance with ASTM E1996 shall be in accordance with ASTM E1886. Impact-resistant glazing and impact protective systems shall comply with the pass/fail criteria of Section 7 of ASTM E1996 based on the missile required by Table 3 or Table 4 of ASTM E1996.

Exception:

Other testing methods and/or performance criteria are permitted to be used when approved.

glazing and impact-protective systems in buildings and structures classified as Occupancy Category I in accordance with Section 103 shall comply with the “enhanced protection” requirements of Table 3 of ASTM E1996. Glazing and impact-protective systems in all other structures shall comply with the “basic protection” requirements of Table 3 of ASTM E1996.

User Note:
The wind zones that are specified in ASTM E1996 for use in determining the applicable missile size for the impact test, have to be adjusted for use with the wind speed maps of this code and the corresponding wind-borne debris regions, see Section C207A.10.3.2.

7A.10.4 Multiple Classifications

A building by definition complies with both the “open” and “partially enclosed” definitions, it shall be classified as “open” building. A building that does not comply with either the “open” or “partially enclosed” definitions shall be classified as an “enclosed” building.

7A.11 Internal Pressure Coefficient

Commentary:
The internal pressure coefficient values in Table 207A.11-1 were obtained from wind tunnel tests (Athapoulos et al. 1979) and full-scale data (Yeatts and Hata 1993). Even though the wind tunnel tests were conducted primarily for low-rise buildings, the internal pressure coefficient values are assumed to be valid for buildings of any height. The values (GC_{pi}) = +0.18 and -0.18 are for enclosed buildings. It is assumed that the building has no dominant opening or openings and that the wind pressure is distributed over the building’s envelope. The internal pressure coefficient values for partially enclosed buildings assume that the building has a dominant opening or openings. For such a building, the internal pressure is dictated by the exterior pressure at the opening and is increased substantially as a result. Net loads, that is, the combination of the internal and exterior pressures, are therefore also significantly increased on the building faces that do not contain the opening. Therefore, higher GC_{pi} values of +0.55 and -0.55 are applicable to this case. These values include a reduction factor to account for the lack of perfect correlation between the internal pressure and the external pressures on the building faces not containing the opening (Irwin 1987 and Best et al. 1996). Taken in isolation, the internal

pressure coefficients can reach values of ± 0.8 (or possibly even higher on the negative side).

For partially enclosed buildings containing a large unpartitioned space, the response time of the internal pressure is increased, and this increase reduces the ability of the internal pressure to respond to rapid changes in pressure at an opening. The gust factor applicable to the internal pressure is therefore reduced. Equation 207A.11-1, which is based on Vickery and Bloxham (1992) and Irwin and Dunn (1994), is provided as a means of adjusting the gust factor for this effect on structures with large internal spaces, such as stadiums and arenas.

Because of the nature of tropical cyclone winds and exposure to debris hazards (Minor and Behr 1993), glazing located below 18 m above the ground level of buildings sited in wind-borne debris regions has a widely varying and comparatively higher vulnerability to breakage from missiles, unless the glazing can withstand reasonable missile loads and subsequent wind loading, or the glazing is protected by suitable shutters. (See Section C207A.10 for discussion of glazing above 18 m. When glazing is breached by missiles, development of higher internal pressure may result, which can overload the cladding or structure if the higher pressure was not accounted for in the design. Breaching of glazing can also result in a significant amount of water infiltration, which typically results in considerable damage to the building and its contents (Surry et al. 1977, Reinhold 1982, and Stubbs and Perry 1993).

The influence of compartmentalization on the distribution of increased internal pressure has not been researched. If the space behind breached glazing is separated from the remainder of the building by a sufficiently strong and reasonably airtight compartment, the increased internal pressure would likely be confined to that compartment. However, if the compartment is breached (e.g., by an open corridor door or by collapse of the compartment wall), the increased internal pressure will spread beyond the initial compartment quite rapidly. The next compartment may contain the higher pressure, or it too could be breached, thereby allowing the high internal pressure to continue to propagate. Because of the great amount of air leakage that often occurs at large hangar doors, designers of hangars should consider utilizing the internal pressure coefficients for partially enclosed buildings in Table 207A.11-1.

207A.11.1 Internal Pressure Coefficients

Internal pressure coefficients, (GC_{pi}), shall be determined from Table 207A.11-1 based on building enclosure classifications determined from Section 207A.10.

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Table 207A.11-1
Internal Pressure Coefficient, (GC_{pi})

Main Wind Force Resisting System and Components and Cladding	All Heights
Enclosed, Partially Enclosed, and Open Buildings	Walls & Roofs
Enclosure Classification	(GC_{pi})
Open Buildings	0.00
Partially Enclosed Buildings	+0.55 -0.55
Enclosed Buildings	+0.18 -0.18

Notes:

1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of (GC_{pi}) shall be used with q_z or q_h as specified.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:
 - i. a positive value of (GC_{pi}) applied to all internal surfaces
 - ii. a negative value of (GC_{pi}) applied to all internal surfaces

207A.11.1.1 Reduction Factor for Large Volume Buildings, R_i

For a partially enclosed building containing a single, unpartitioned large volume, the internal pressure coefficient, (GC_{pi}) , shall be multiplied by the following reduction factor, R_i :

$$R_i = 1.0 \text{ or}$$

$$R_i = 0.5 \left(1 + \frac{1}{\sqrt{1 + \frac{V_i}{7 A_{og}}}} \right) < 1.0 \quad (207A.11-1)$$

where

- A_{og} = total area of openings in the building envelope (walls and roof), in m^2
 V_i = unpartitioned internal volume, in m^3

207B Wind Loads On Buildings—MWFRS (Directional Procedure)

Commentary:

The Directional Procedure is the former "buildings of all heights" provision in Method 2 of NSCP 2010 (ASCE 7-05) for MWFRS. A simplified method based on this Directional Procedure is provided for buildings up to 49 m in height. The Directional Procedure is considered the traditional approach in that the pressure coefficients reflect the actual loading on each surface of the building as a function of wind direction, namely, winds perpendicular or parallel to the ridge line.

207B.1 Scope

207B.1.1 Building Types

This chapter applies to the determination of MWFRS wind loads on enclosed, partially enclosed, and open buildings of all heights using the Directional Procedure.

1. Part 1 applies to buildings of all heights where it is necessary to separate applied wind loads onto the windward, leeward, and side walls of the building to properly assess the internal forces in the MWFRS members.
2. Part 2 applies to a special class of buildings designated as enclosed simple diaphragm buildings, as defined in Section 207A.2, with $h \leq 48$ m.

207B.1.2 Conditions

A building whose design wind loads are determined in accordance with this chapter shall comply with all of the following conditions:

1. The building is a regular-shaped building or structure as defined in Section 207A.2.
2. The building does not have response characteristics making it subject to across-wind loading, vortex shedding, instability due to galloping or flutter; or it does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

207B.1.3 Limitations

The provisions of this chapter take into consideration the load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings. Buildings not meeting the requirements of Section 207B.1.2, or having unusual shapes or response characteristics shall be designed using recognized literature documenting such wind load effects or shall use the wind tunnel procedure specified in Section 207F.

207B.1.4 Shielding

There shall be no reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

Part 1: Enclosed, Partially Enclosed, and Open Buildings of All Heights**207B.2 General Requirements**

The steps to determine the wind loads on the MWFRS for enclosed, partially enclosed and open buildings of all heights are provided in Table 207B.2-1.

User Note:

Use Part 1 of Section 207B to determine wind pressures on the MWFRS of enclosed, partially enclosed or an open building with any general plan shape, building height or roof geometry that matches the figures provided. These provisions utilize the traditional "all heights" method (Directional Procedure) by calculating wind pressures using specific wind pressure equations applicable to each building surface.

207B.2.1 Wind Load Parameters Specified in Section 207A

The following wind load parameters shall be determined in accordance with Section 207A:

- Basic Wind Speed, V (Section 207A.5)
- Wind directionality factor, K_d (Section 207A.6)
- Exposure category (Section 207A.7)
- Topographic factor, K_{zt} (Section 207A.8)
- Gust-effect factor (Section 207A.9)
- Enclosure classification (Section 207A.10)
- Internal pressure coefficient, (GC_{pi}) (Section 207A.11)

207B.3 Velocity Pressure**207B.3.1 Velocity Pressure Exposure Coefficient**

Based on the exposure category determined in Section 207A.7.3, a velocity pressure exposure coefficient K_z or K_h , as applicable, shall be determined from Table 207B.3-1. For a site located in a transition zone between exposure categories that is near to a change in ground surface roughness, intermediate values of K_z or K_h , between those shown in Table 207B.3-1 are permitted provided that they are determined by a rational analysis method defined in the recognized literature.

Table 207B.2-1
Steps to Determine MWFRS Wind
Loads for Enclosed, Partially Enclosed and Open
Buildings of All Heights

Step 1:	Determine risk category of building or other structure, see Table 103-1
Step 2:	Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C
Step 3:	Determine wind load parameters: ➤ Wind directionality factor, K_d, see Section 207A.6 and Table 207A.6-1 ➤ Exposure category, see Section 207A.7 ➤ Topographic factor, K_{zt}, see Section 207A.8 and Table 207A.8-1 ➤ Gust Effect Factor, G, see Section 207A.9 ➤ Enclosure classification, see Section 207A.10 ➤ Internal pressure coefficient, (GC_{pi}), see Section 207A.11 and Table 207A.11-1
Step 4:	Determine velocity pressure exposure coefficient, K_z or K_h , see Table 207B.3-1
Step 5:	Determine velocity pressure q_z or q_h Equation 207B.3-1
Step 6:	Determine external pressure coefficient, C_p or C_N ➤ Figure 207B.4-1 for walls and flat, gable, hip, monoslope or mansard roofs ➤ Figure 207B.4-2 for domed roofs ➤ Figure 207B.4-3 for arched roofs ➤ Figure 207B.4-4 for monoslope roof, open building ➤ Figure 207B.4-5 for pitched roof, open building ➤ Figure 207B.4-6 for troughed roof, open building ➤ Figure 207B.4-7 for along-ridge/valley wind load case for monoslope, pitched or troughed roof, open building
Step 7:	Calculate wind pressure, p , on each building surface ➤ Equation 207B.4-1 for rigid buildings ➤ Equation 207B.4-2 for flexible buildings ➤ Equation 207B.4-3 for open buildings

Commentary:

The velocity pressure exposure coefficient K_z can be obtained using the equation:

$$K_z = \begin{cases} 2.01 \left(\frac{z}{z_g} \right)^{2/\alpha} & \text{For } 4.57\text{m} \leq z \leq z_g \quad (\text{C207B.3-1}) \\ 2.01 \left(\frac{4.57}{z_g} \right)^{2/\alpha} & \text{For } z < 4.57\text{m} \quad (\text{C207B.3-2}) \end{cases}$$

in which values of α and z_g are given in Table 207A.9-1. These equations are now given in Tables 207B.3-1, 207C.3-1, 207D.3-1, and 207E.3-1 to aid the user.

Changes were implemented in NSCP 2001 (ASCE 7-98), including truncation of K_z values for Exposures A and B below heights of 30 m and 9 m, respectively, applicable to Components and Cladding and the Envelope Procedure. Exposure A was eliminated in the 2002 edition of ASCE 7.

In the NSCP 2010 (ASCE 7-05) standard, the K_z expressions were unchanged from NSCP 2001 (ASCE 7-98). However, the possibility of interpolating between the standard exposures using a rational method was added in the NSCP 2010 (ASCE 7-05) edition. One rational method is provided in the following text.

To a reasonable approximation, the empirical exponent α and gradient height z_g in the preceding expressions (Equations C207B.3-1 and C207B.3-2) for exposure coefficient K_z may be related to the roughness length z_0 (where z_0 is defined in Section C207A.7) by the relations

$$\alpha = c_1 z_0^{-0.133} \quad (\text{C207B.3-3})$$

and

$$z_g = c_2 z_0^{0.125} \quad (\text{C207B.3-4})$$

where

Units of z_0, z_g	c_1	c_2
m	5.65	450

The preceding relationships are based on matching the ESDU boundary layer model (Harris and Deaves 1981 and ESDU 1990 and 1993) empirically with the power law relationship in Equations C207B.3-1 and C207B.3-2, the ESDU model being applied at latitude 35° with a gradient

wind of 75 m/s. If z_0 has been determined for a particular upwind fetch, Equations C207B.3-1 through C207B.3-4 can be used to evaluate K_z . The correspondence between z_0 and the parameters α and z_g implied by these relationships does not align exactly with that described in the commentary to ASCE 7-95 and 7-98. However, the differences are relatively small and not of practical consequence. The ESDU boundary layer model has also been used to derive the following simplified method (Irwin 2006) of evaluating K_z following a transition from one surface roughness to another. For more precise estimates the reader is referred to the original ESDU model (Harris and Deaves 1981 and ESDU 1990 and 1993).

In uniform terrain, the wind travels a sufficient distance over the terrain for the planetary boundary layer to reach an equilibrium state. The exposure coefficient values in Table 207B.3-1 are intended for this condition. Suppose that the site is a distance x miles downwind of a change in terrain. The equilibrium value of the exposure coefficient at height z for the terrain roughness downwind of the change will be denoted by K_{zd} , and the equilibrium value for the terrain roughness upwind of the change will be denoted by K_{zu} . The effect of the change in terrain roughness on the exposure coefficient at the site can be represented by adjusting K_{zd} by an increment ΔK , thus arriving at a corrected value K_z for the site.

$$K_z = K_{zd} + \Delta K \quad (\text{C207B.3-5})$$

In this expression ΔK is calculated using:

$$\Delta K = (K_{33,u} - K_{33,d}) \frac{K_{zd}}{K_{33,d}} F_{\Delta K}(x) \quad (\text{C207B.3-6})$$

$$|\Delta K| \leq |K_{zu} - K_{zd}|$$

where $K_{33,d}$ and $K_{33,u}$ are respectively the downwind and upwind equilibrium values of exposure coefficient at 10 m height, and the function $F_{\Delta K}(x)$ is given by:

$$F_{\Delta K}(x) = \log_{10} \left(\frac{x_1}{x} \right) / \log_{10} \left(\frac{x_1}{x_0} \right) \quad (\text{C207B.3-7})$$

For $x_0 < x < x_1$

$$F_{\Delta K}(x) = 1 \quad \text{for } x < x_0$$

$$F_{\Delta K}(x) = 0 \quad \text{for } x > x_1$$

In the preceding relationships

$$x_0 = c_3 \times 10^{-(K_{33,d} - K_{33,u})^2 - 2.3} \quad (\text{C207B.3-8})$$

The constant $c_3 = 1.0$ km. The length $x_1 = 10$ km for $K_{33,d} < K_{33,u}$ (wind going from smoother terrain upwind to rougher terrain downwind) or $x_1 = 100$ km for $K_{33,d} > K_{33,u}$ (wind going from rougher terrain upwind to smoother terrain downwind).

The above description is in terms of a single roughness change. The method can be extended to multiple roughness changes. The extension of the method is best described by an example. Figure C207B.3-1 shows wind with an initial profile characteristic of Exposure D encountering an expanse of B roughness, followed by a further expanse of D roughness and then some more B roughness again before it arrives at the building site. This situation is representative of wind from the sea flowing over an outer strip of land, then a coastal waterway, and then some suburban roughness before arriving at the building site. The above method for a single roughness change is first used to compute the profile of K_z at station 1 in Figure C207B.3-1. Call this profile $K_z^{(1)}$. The value of ΔK for the transition between stations 1 and 2 is then determined using the equilibrium value of $K_{33,u}$ for the roughness immediately upwind of station 1, i.e., as though the roughness upwind of station 1 extended to infinity. This value of ΔK is then added to the equilibrium value $K_z^{(2)}$ of the exposure coefficient for the roughness between stations 1 and 2 to obtain the profile of K_z at station 2, which we will call $K_z^{(2)}$. Note however, that the value of $K_z^{(2)}$ in this way cannot be any lower than $K_z^{(1)}$. The process is then repeated for the transition between stations 2 and 3. Thus, ΔK for the transition from station 2 to station 3 is calculated using the value of $K_{33,u}$ for the equilibrium profile of the roughness immediately upwind of station 2, and the value of $K_{33,d}$ for the equilibrium profile of the roughness downwind of station 2. This value of ΔK is then added to $K_z^{(2)}$ to obtain the profile $K_z^{(3)}$ at station 3, with the limitation that the value of $K_z^{(3)}$ cannot be any higher than $K_z^{(2)}$.

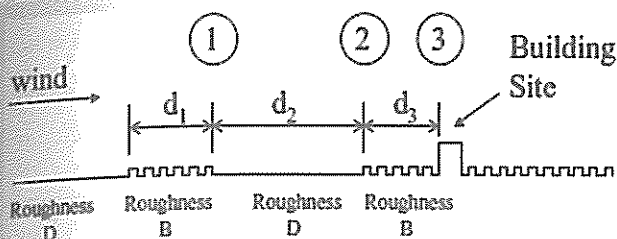


Figure C207B.3-1
Multiple Roughness Changes Due to
Coastal Waterway

Example 1, single roughness change: Suppose the building is 20 m high and its local surroundings are suburban with a roughness length $z_0 = 0.3$ m. However, the site is 0.6 km downwind of the edge of the suburbs, beyond which the open terrain is characteristic of open country with $z_0 = 0.02$ m. From Equations C207B.3-1, C207B.3-3, and C207B.3-4, for the open terrain

$$\alpha = c_1 z_0^{-0.133} = \frac{6.62}{\times 0.066^{-0.133}} = 9.5$$

$$z_g = c_2 z_0^{0.125} = \frac{1,273}{\times 0.066^{0.125}} = 276 \text{ m}$$

Therefore, applying Equation C207B.3-1 at 20 m and 10 m heights,

$$K_{zu} = 2.01 \left(\frac{66}{906} \right)^{2/9.5} = 1.16 \text{ and}$$

$$K_{33,u} = 2.01 \left(\frac{33}{906} \right)^{2/9.5} = 1.0$$

Similarly, for the suburban terrain

$$\alpha = c_1 z_0^{-0.133} = \frac{6.62 \times 1.0^{-0.133}}{} = 6.62$$

$$z_g = c_2 z_0^{0.125} = \frac{1,273 \times 1.0^{0.125}}{} = 388 \text{ m}$$

Therefore

$$K_{zd} = 2.01 \left(\frac{66}{1,273} \right)^{2/6.19} = 0.77 \text{ and}$$

$$K_{33,d} = 2.01 \left(\frac{33}{1,273} \right)^{2/6.62} = 0.67$$

From Equation C207B.3-8

$$x_0 = c_3 \times 10^{-(K_{33,d} - K_{33,u})^2 - 2.3}$$

$$= 0.621 \times 10^{-(0.62 - 1.00)^2 - 2.3}$$

$$= 0.00241 \text{ mi}$$

From Equation C207B.3-7

$$F_{\Delta K}(x) = \log_{10} \left(\frac{6.21}{0.36} \right) / \log_{10} \left(\frac{6.21}{0.00241} \right) = 0.36$$

Therefore from Equation C207B.3-6

$$\Delta K = (1.00 - 0.67) \frac{0.82}{0.67} 0.36 = 0.15$$

Note that because $|\Delta K|$ is 0.15, which is less than the 0.38 value of $|K_{33,u} - K_{33,d}|$, 0.15 is retained. Finally, from Equation C207B.3-5, the value of K_z is:

$$K_z = K_{zd} + \Delta K = 0.82 + 0.15 = 0.97$$

Because the value 0.97 for K_z lies between the values 0.88 and 1.16, which would be derived from Table 207B.3-1 for Exposures B and C respectively, it is an acceptable interpolation. If it falls below the Exposure B value, then the Exposure B value of K_z is to be used. The value $K_z = 0.97$ may be compared with the value 1.16 that would be required by the simple 792 m fetch length requirement of Section 207A.7.3.

The most common case of a single roughness change where an interpolated value of K_z is needed is for the transition from Exposure C to Exposure B, as in the example just described. For this particular transition, using the typical values of z_0 of 0.02 m and 0.3 m, the preceding formulae can be simplified to:

$$K_z = K_{zd} \left(1 + 0.146 \log_{10} \left(\frac{6.21}{x} \right) \right) \quad (\text{C207B.3-9})$$

$$K_{zB} \leq K_z \leq K_{zC}$$

where x is in miles, and K_{zd} is computed using $\alpha = 6.62$. K_{zB} and K_{zC} are the exposure coefficients in the standard Exposures C and B. Figure C207B.3-2 illustrates the transition from terrain roughness C to terrain roughness B from this expression. Note that it is acceptable to use the typical z_0 rather than the lower limit for Exposure B in deriving this formula because the rate of transition of the wind profiles is dependent on average roughness over significant distances, not local roughness anomalies. The potential effects of local roughness anomalies, such as

parking lots and playing fields, are covered by using the standard Exposure B value of exposure coefficient, K_{zB} , as a lower limit to the calculated value of K_z .

Example 2: Multiple Roughness Change Suppose we have a coastal waterway situation as illustrated in Figure C207B.3-1, where the wind comes from open sea with roughness type D, for which we assume $z_0 = 0.003$ m, and passes over a strip of land 1.61 km wide, which is covered in buildings that produce typical B type roughness, i.e. $z_0 = 0.3$ m. It then passes over a 3.22-km wide strip of coastal waterway where the roughness is again characterized by the open water value $z_0 = 0.003$ m. It then travels over 0.16 km of roughness type B ($z_0 = 0.3$ m) before arriving at the site, station 3 in Figure C207B.3-1,

where the exposure coefficient is required at the 15-m height. The exposure coefficient at station 3 at 15 m height is calculated as shown in Table C207B.3-1.

The value of the exposure coefficient at 15 m at station 3 is seen from the table to be 1.067. This is above that for Exposure B, which would be 0.81, but well below that for Exposure D, which would be 1.27, and similar to that for Exposure C, which would be 1.09.

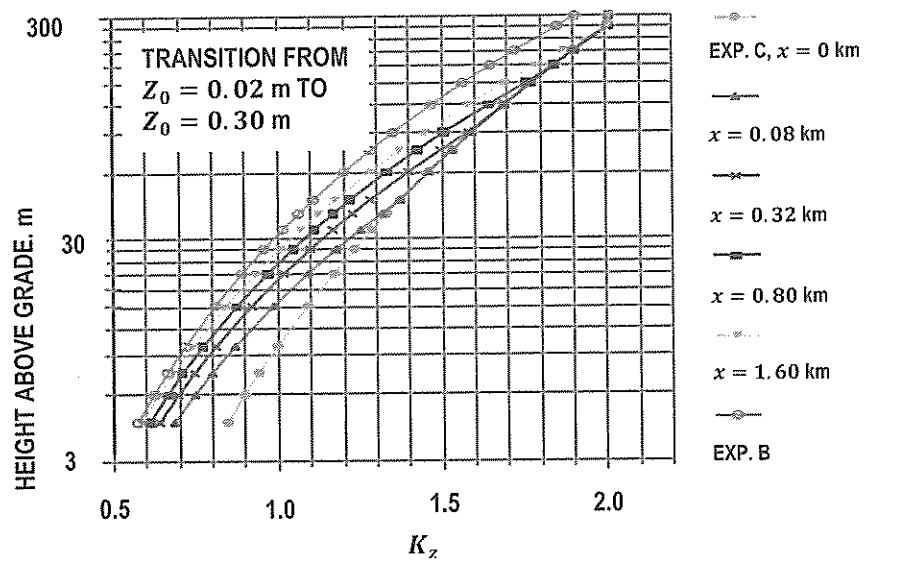


Figure C207B.3-2
Transition from Terrain Roughness C to Terrain Roughness B, Equation C207B.3.1-9

Table C207B.3-1
Tabulated Exposure Coefficients

Transition from sea to station 1	$K_{10,u}$ 1.215	$K_{10,d}$ 0.667	$K_{15,d}$ 0.758	$F_{\Delta K}$ 0.220	ΔK_{15} 0.137	$K_z^{(1)}$ 0.895
Transition from sea to station 2	$K_{10,u}$ 0.667	$K_{10,d}$ 1.215	$K_{15,d}$ 1.215	$F_{\Delta K}$ 0.324	ΔK_{15} -0.190	$K_z^{(2)}$ 1.111
Transition from sea to station 3	$K_{10,u}$ 1.215	$K_{10,d}$ 0.667	$K_{15,d}$ 0.667	$F_{\Delta K}$ 0.498	ΔK_{15} 0.310	$K_z^{(3)}$ 1.067

Note: The equilibrium values of the exposure coefficients, $K_{10,u}$, $K_{10,d}$ and $K_{15,d}$ (downwind value of K_z at 15 m), were calculated from Equation C207B-1 using α and z_g values obtained from Equations C207B-3 and C207B-4 with the roughness values given. Then $F_{\Delta K}$ is calculated using Equations C207B-7 and C207B-8, and then the value of ΔK at 15 m height, ΔK_{15} , is calculated from Equation C207B-6. Finally, the exposure coefficient at 15 m at station i , $K_{15}^{(i)}$, is obtained from Equation C207B-5.

207B.3.2 Velocity Pressure

Velocity pressure, q_z , evaluated at height z shall be calculated by the following equation:

$$q_z = 0.613 K_z K_{zt} K_d V^2 (\text{N/m}^2); V \text{ in m/s} \quad (207B.3-1)$$

where

- K_d = wind directionality factor, see Section 207A.6
- K_z = velocity pressure exposure coefficient, see Section 207B.3.1
- K_{zt} = topographic factor defined, see Section 207A.8.2
- V = basic wind speed, see Section 207A.5
- q_z = velocity pressure calculated using Equation 207B.3-1 at height z
- q_{zt} = velocity pressure calculated using Equation 207B.3-1 at mean roof height h

The numerical coefficient 0.613 shall be used except where sufficient climatic data are available to justify the selection of a different value of this coefficient for a design application.

Commentary:

The basic wind speed is converted to a velocity pressure q_z in (N/m^2) at height z by the use of Equation 207B.3-1.

The constant 0.613 reflects the mass density of air for the standard atmosphere, that is, temperature of 15 °C and sea level pressure of 101.325 kPa, and dimensions associated with wind speed in m/s. The constant is obtained as follows:

$$\begin{aligned} \text{constant} &= 1/2[(1.225 \text{ kg/m}^3)/(9.81 \text{ m/s}^2)] \times \\ &= [(m/s)]^2 [9.81 \text{ N/kg}] \\ &= 0.613 \end{aligned}$$

207B.4 Wind Loads—Main Wind Force-Resisting System

207B.4.1 Enclosed and Partially Enclosed Rigid Buildings

Design wind pressures for the MWFRS of buildings of all heights shall be determined by the following equation:

$$p = qGC_p - q_i(GC_{pi}) \text{ (N/m}^2\text{)} \quad (207B.4-1)$$

where

q = q_z for windward walls evaluated at height z above the ground

q = q_h for leeward walls, side walls, and roofs, evaluated at height h

q_i = q_h for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings

q_i = q_z for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact resistant covering shall be treated as an opening in accordance with Section 207A.10.3.

For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$)

G = gust-effect factor, see Section 207A.9

C_p = external pressure coefficient from Figures 207B.4-1, 207B.4-2 and 207B.4-3

(GC_{pi}) = internal pressure coefficient from Table 207A.11-1

q and q_i shall be evaluated using exposure defined in Section 207A.7.3. Pressure shall be applied simultaneously on windward and leeward walls and on roof surfaces as defined in Figures 207B.4-1, 207B.4-2 and 207B.4-3.

The numerical constant of 0.613 should be used except where sufficient weather data are available to justify a different value of this constant for a specific design application. The mass density of air will vary as a function of altitude, latitude, temperature, weather, and season. Average and extreme values of air density are given in Table C207B.3-2.

Table 207B.3-1
Velocity Pressure Exposure Coefficients, K_h and K_z
Main Wind Force Resisting System – Part 1

Height above ground level, z (m)	Exposure		
	B	C	D
0 - 4.5	0.57	0.85	1.03
6.0	0.62	0.90	1.08
7.5	0.66	0.94	1.12
9.0	0.70	0.98	1.16
12.0	0.76	1.04	1.22
15.0	0.81	1.09	1.27
18.0	0.85	1.13	1.31
21.0	0.89	1.17	1.34
24.0	0.93	1.21	1.38
27.0	0.96	1.24	1.40
30.0	0.99	1.26	1.43
36.0	1.04	1.31	1.48
42.0	1.09	1.36	1.52
48.0	1.13	1.39	1.55
54.0	1.17	1.43	1.58
60.0	1.20	1.46	1.61
75.0	1.28	1.53	1.68
90.0	1.35	1.59	1.73
105.0	1.41	1.64	1.78
120.0	1.47	1.69	1.82
135.0	1.52	1.73	1.86
150.0	1.56	1.77	1.89

Notes:

1. The velocity pressure exposure coefficient K_z , may be determined from the following formula:

For $4.5 \text{ m} \leq z \leq z_g$

For $z < 4.5 \text{ m}$

$$K_z = 2.01(z/z_g)^{2/\alpha} \quad K_z = 2.01(4.5/z_g)^{2/\alpha}$$

2. α and z_g are tabulated in Table 207A.9.1.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposure categories are defined Section 207A.7.

Commentary:

Loads on Main Wind-Force Resisting Systems: In Equations 207B.4-1 and 207B.4-2, a velocity pressure term q_i appears that is defined as the "velocity pressure for internal pressure determination." The positive internal pressure is dictated by the positive exterior pressure on the windward face at the point where there is an opening. The positive exterior pressure at the opening is governed by the value of q at the level of the

opening, not q_h . For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$). For low buildings this does not make much difference, but for the example of a 90-m tall building in Exposure B with a highest opening at 18 m, the difference between q_{90} and q_{18} represents a 59 percent increase in internal pressure. This difference is unrealistic and represents an unnecessary degree of conservatism. Accordingly, $q_i = q_h$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, with glazing that is not impact resistant or protected with an impact protective system, q_i should be treated on the assumption there will be an opening.

Figure 207B.4-1. The pressure coefficients for MWFRSs are separated into two categories:

1. Directional Procedure for buildings of all heights (Figure 207B.4-1) as specified in Section 207B for buildings meeting the requirements specified therein.
2. Envelope Procedure for low-rise buildings having a height less than or equal to 18 m (Figure 207C.4-1) as specified in Section 207C for buildings meeting the requirements specified therein.

In generating these coefficients, two distinctly different approaches were used. For the pressure coefficients given in Figure 207B.4-1, the more traditional approach was followed and the pressure coefficients reflect the actual loading on each surface of the building as a function of wind direction; namely, winds perpendicular or parallel to the ridge line. Observations in wind tunnel tests show that areas of very low negative pressure and even slightly positive pressure can occur in all roof structures, particularly as the distance from the windward edge increases and the wind streams reattach to the surface. These pressures can occur even for relatively flat or low slope roof structures. Experience

and judgment from wind tunnel studies have been used to specify either zero or slightly negative pressures (-0.18) depending on the negative pressure coefficient. These values require the designer to consider a zero or slightly positive net wind pressure in the load combinations of Section 203.

Table C207B.3-2
Ambient Air Density Values for Various Altitudes

Altitude Meters	Ambient Air Temperature		
	Minimum (kg/m ³)	Average (kg/m ³)	Maximum (kg/m ³)
0	1.1392	1.2240	1.3152
305	1.1088	1.1872	1.2720
610	1.0800	1.1520	1.2288
914	1.0512	1.1184	1.1888
1000	1.0432	1.1088	1.1776
1219	1.0240	1.0848	1.1488
1524	0.9984	1.0544	1.1120
1829	0.9728	1.0224	1.0752
2000	0.9584	1.0064	1.0560
2134	0.9472	0.9920	1.0400
2438	0.9232	0.9632	1.0048
2743	0.8976	0.9344	0.9712
3000	0.8784	0.9104	0.9456
3048	0.8752	0.9072	0.9408

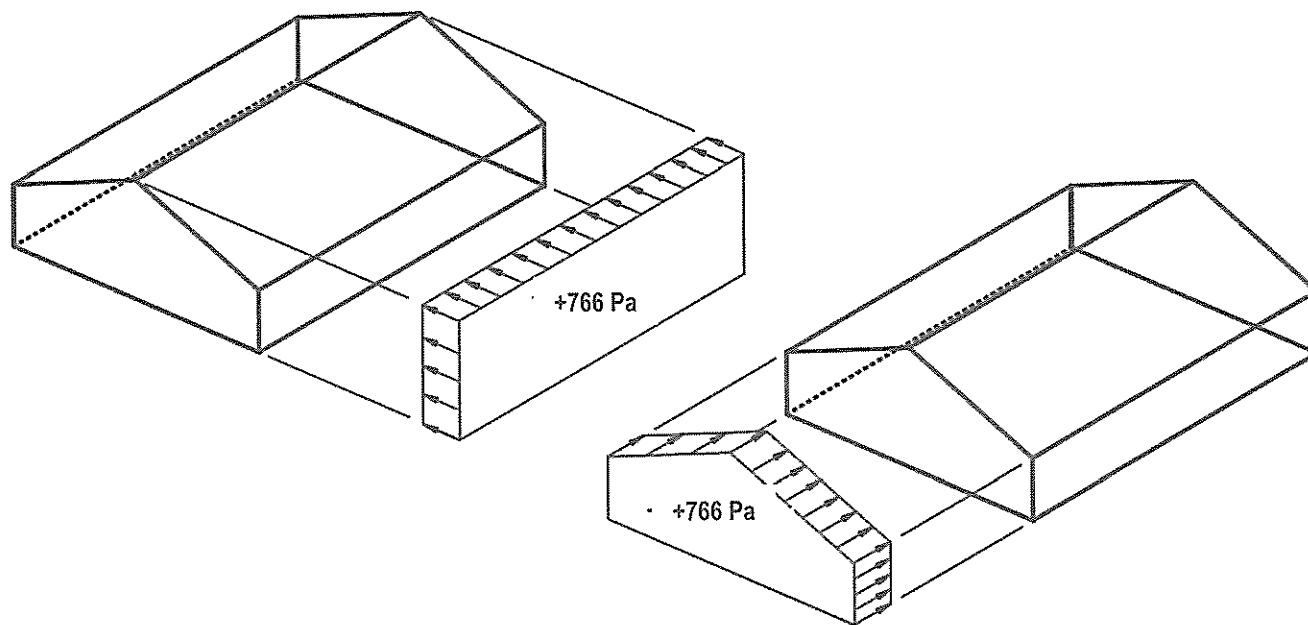


Figure C207B.4-1
Application of Minimum Wind Load

Figure 207B.4-2. Frame loads on dome roofs are adapted from the Eurocode (1995). The loads are based on data obtained in a modeled atmospheric boundary-layer flow that does not fully comply with requirements for wind-tunnel testing specified in this code (Blessman 1971). Loads for three domes ($h_D/D = 0.5, f/D = 0.5$), ($h_D/D = 0, f/D = 0.5$), and ($h_D/D = 0, f/D = 0.33$) are roughly consistent with data of Taylor (1991), who used an atmospheric boundary layer as required in this code. Two load cases are defined, one of which has a linear variation of pressure from A to B as in the Eurocode (1995) and one in which the pressure at A is held constant from 0° to 25° ; these two cases are based on comparison of the Eurocode provisions with Taylor (1991). Case A (the Eurocode calculation) is necessary in many cases to define maximum uplift. Case B is necessary to properly define positive pressures for some cases, which cannot be isolated with current information, and which result in maximum base shear. For domes larger than 60 m in diameter the designer should consider use of wind-tunnel testing. Resonant response is not considered in these provisions. Wind-tunnel testing should be used to consider resonant response. Local bending moments in the dome shell may be larger than predicted by this method due to the difference between instantaneous local pressure distributions and those predicted by Figure 207B.4-2. If the dome is supported on vertical walls directly below, it is appropriate to consider the walls as a “chimney” using Figure 207D.5-1.

Figure 207B.4-3. The pressure and force coefficient values in these tables are unchanged from ANSI A58.1-1972. The coefficients specified in these tables are based on wind-tunnel tests conducted under conditions of uniform flow and low turbulence, and their validity in turbulent boundary-layer flows has yet to be completely established. Additional pressure coefficients for conditions not specified herein may be found in SIA (1956) and ASCE (1961).

207B.4.2 Enclosed and Partially Enclosed Flexible Buildings

Design wind pressures for the MWFRS of flexible buildings shall be determined from the following equation:

$$p = qG_fC_p - q_i(GC_{pi}) \text{ (N/m}^2\text{)} \quad (207B.4-2)$$

where q , q_i , C_p , and (GC_{pi}) are as defined in Section 207B.4.1 and G_f (gust-effect factor) is determined in accordance with Section 207A.9.5.

207B.4.3 Open Buildings with Monoslope, Pitched, or Troughed Free Roofs

The net design pressure for the MWFRS of open buildings with monoslope, pitched, or troughed roofs shall be determined by the following equation:

$$p = q_h G C_N \quad (\text{N/m}^2) \quad (207\text{B.4-3})$$

where

- q_h = velocity pressure evaluated at mean roof height h using the exposure as defined in Section 207A.7.3 that results in the highest wind loads for any wind direction at the site
- G = gust-effect factor from Section 207A.9
- C_N = net pressure coefficient determined from Figures 207B.4-4 through 207B.4-7

Net pressure coefficients, C_N , include contributions from top and bottom surfaces. All load cases shown for each roof angle shall be investigated. Plus and minus signs signify pressure acting toward and away from the top surface of the roof, respectively.

For free roofs with an angle of plane of roof from horizontal θ less than or equal to 5° and containing fascia panels, the fascia panel shall be considered an inverted parapet. The contribution of loads on the fascia to the MWFRS loads shall be determined using Section 207B.4.5 with q_p equal to q_h .

Commentary:

Figures 207B.4-4 through 207B.4-6 and 207E.8-1 through 207E.8-3 are presented for wind loads on MWFRSs and components and cladding of open buildings with roofs as shown, respectively. This work is based on the Australian Standard AS1170.2-2000, Part 2: Wind Actions, with modifications to the MWFRS pressure coefficients based on recent studies (Altman and Uematsu and Stathopoulos 2003).

Two load cases, A and B, are given in Figures 207B.4-4 through 207B.4-6. These pressure distributions provide loads that envelop the results from detailed wind-tunnel measurements of simultaneous normal forces and moments. Application of both load cases is required to envelop the combinations of maximum normal forces and moments that are appropriate for the particular roof shape and blockage configuration.

The roof wind loading on open building roofs is highly dependent upon whether goods or materials are stored

under the roof and restrict the wind flow. Restricting the flow can introduce substantial upward acting pressures on the bottom surface of the roof, thus increasing the resultant uplift load on the roof. Figures 207B.4-4 through 207B.4-6 and 207E.8-1 through 207E.8-3 offer the designer two options. Option 1 (clear wind flow) implies little (less than 50 percent) or no portion of the cross-section below the roof is blocked. Option 2 (obstructed wind flow) implies that a significant portion (more than 75 percent is typically referenced in the literature) of the cross-section is blocked by goods or materials below the roof. Clearly, values would change from one set of coefficients to the other following some sort of smooth, but as yet unknown, relationship. In developing the provisions included in this code, the 50 percent blockage value was selected for Option 1, with the expectation that it represents a somewhat conservative transition. If the designer is not clear about usage of the space below the roof or if the usage could change to restrict free air flow, then design loads for both options should be used.

207B.4.4 Roof Overhangs

The positive external pressure on the bottom surface of windward roof overhangs shall be determined using $C_p = 0.8$ and combined with the top surface pressures determined using Figure 207B.4-1.

207B.4.5 Parapets

The design wind pressure for the effect of parapets on MWFRS of rigid or flexible buildings with flat, gable, or hip roofs shall be determined by the following equation:

$$p_p = q_p (G C_{pn}) \quad (\text{N/m}^2) \quad 207\text{B.4-4}$$

where

- p_p = combined net pressure on the parapet due to the combination of the net pressures from the front and back parapet surfaces. Plus (and minus) signs signify net pressure acting toward (and away from) the front (exterior) side of the parapet
- q_p = velocity pressure evaluated at the top of the parapet
- $(G C_{pn})$ = combined net pressure coefficient
 - = +1.5 for windward parapet
 - = -1.0 for leeward parapet

207B.4.6 Design Wind Load Cases

The MWFRS of buildings of all heights, whose wind loads have been determined under the provisions of this chapter, shall be designed for the wind load cases as defined in Figure 207B.4-8.

Exception:

Buildings meeting the requirements of Section D1.1 of Appendix D, ASCE 7-10 need only be designed for Case 1 and Case 3 of Figure 207B.4-8.

The eccentricity e for rigid structures shall be measured from the geometric center of the building face and shall be considered for each principal axis (e_x, e_y). The eccentricity e for flexible structures shall be determined from the following equation and shall be considered for each principal axis (e_x, e_y):

$$e = \frac{e_Q + 1.71I_z \sqrt{(g_Q Q e_Q)^2 + (g_R R e_R)^2}}{1.71I_z \sqrt{(g_Q Q e_Q)^2 + (g_R R e_R)^2}} \quad 207B.4-5$$

where

e_Q = eccentricity e as determined for rigid structures in Figure 207B.4-8

e_R = distance between the elastic shear center and center of mass of each floor

I_z , g_Q , Q , g_R , and R shall be as defined in Section 207A.9

The sign of the eccentricity e shall be plus or minus, whichever causes the more severe load effect.

Commentary:

Wind tunnel research (Isyumov 1983, Boggs et al. 2000, Isyumov and Case 2000, and Xie and Irwin 2000) has shown that torsional load is caused by non-uniform pressure on the different faces of the building from wind flow around the building, interference effects of nearby buildings and terrain, and by dynamic effects on more flexible buildings. Load Cases 2 and 4 in Figure 207B.4-8 specifies the torsional loading to 15 percent eccentricity under 75 percent of the maximum wind shear for Load Case 2. Although this is more in line with wind tunnel experience on square and rectangular buildings with aspect ratios up to about 2.5, it may not cover all cases, even for symmetric and common building shapes where larger torsions have been observed. For example, wind tunnel studies often show an eccentricity of 5 percent or

more under full (not reduced) base shear. The designer may wish to apply this level of eccentricity at full wind loading for certain more critical buildings even though it is not required by the code. The present more moderate torsional load requirements can in part be justified by the fact that the design wind forces tend to be upper-bound for most common building shapes.

In buildings with some structural systems, more severe loading can occur when the resultant wind load acts diagonally to the building. To account for this effect and the fact that many buildings exhibit maximum response in the across-wind direction (the standard currently has no analytical procedure for this case), a structure should be capable of resisting 75 percent of the design wind load applied simultaneously along each principal axis as required by Case 3 in Figure 207B.4-8.

For flexible buildings, dynamic effects can increase torsional loading. Additional torsional loading can occur because of eccentricity between the elastic shear center and the center of mass at each level of the structure. Equation 207B.4-5 accounts for this effect.

It is important to note that significant torsion can occur on low-rise buildings also (Isyumov and Case 2000) and, therefore, the wind loading requirements of Section 207B.4.6 are now applicable to buildings of all heights.

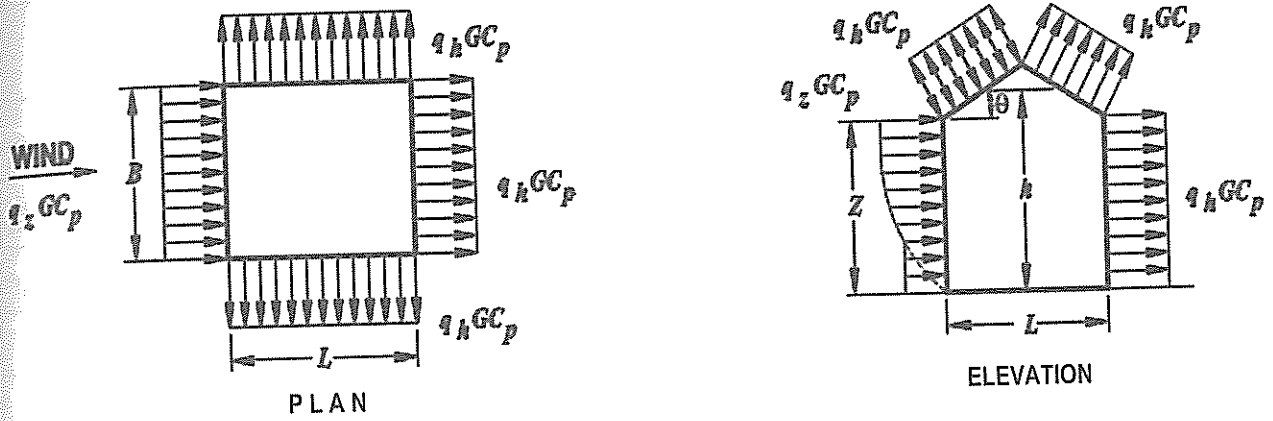
As discussed in Section 207F, the wind tunnel procedure should always be considered for buildings with unusual shapes, rectangular buildings with larger aspect ratios, and dynamically sensitive buildings. The effects of torsion can more accurately be determined for these cases and for the more normal building shapes using the wind tunnel procedure.

207B.4.7 Minimum Design Wind Loads

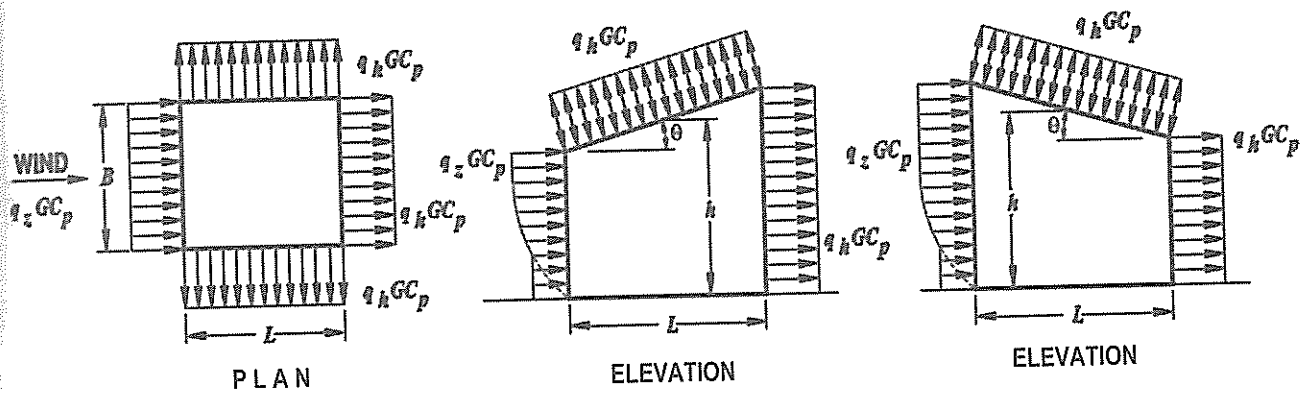
The wind load to be used in the design of the MWFRS for an enclosed or partially enclosed building shall not be less than 0.77 kN/m^2 multiplied by the wall area of the building and 0.38 kN/m^2 multiplied by the roof area of the building projected onto a vertical plane normal to the assumed wind direction. Wall and roof loads shall be applied simultaneously. The design wind force for open buildings shall be not less than 0.77 kN/m^2 multiplied by the area A_f .

Commentary:

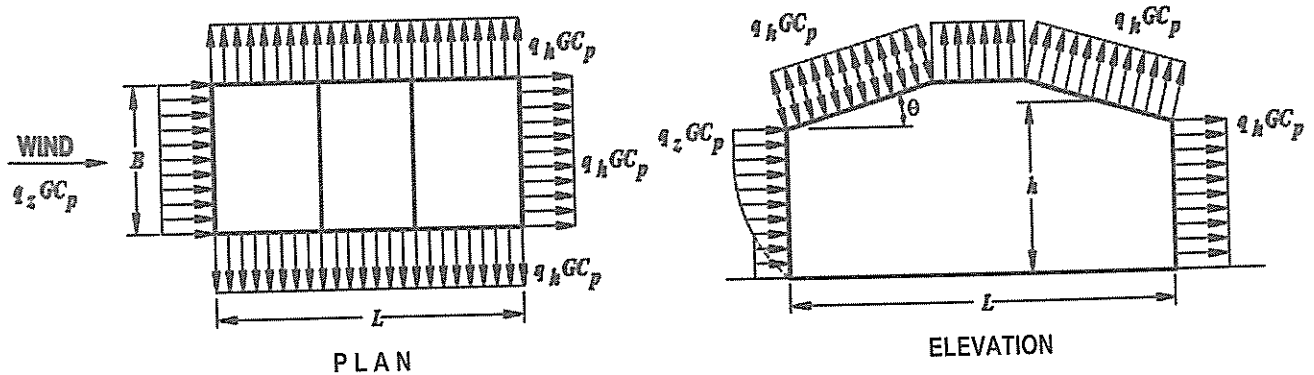
This section specifies a minimum wind load to be applied horizontally on the entire vertical projection of the building as shown in Figure C207B.4-1. This load case is to be applied as a separate load case in addition to the normal load cases specified in other portions of this chapter.



GABLE, HIP ROOF



MONOSLOPE ROOF (NOTE 4)



MANSARD ROOF (NOTE 8)

Figure 207B.4-1
External Pressure Coefficients, C_p , Walls and Roofs Enclosed, Partially Enclosed Buildings

Wall Pressure Coefficients, C_p			
Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥ 4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p , for use with q_h													
Wind Direction	Windward									Leeward			
	Angle, θ (degrees)									Angle, θ (degrees)			
	h/L	10	15	20	25	30	35	45	≥ 60	10	15	≥ 20	
Normal to Ridge for $\theta \geq 10^\circ$		-0.7	-0.5	-0.3	-0.2	-0.2	0.0*						
	≤ 0.25	-0.18	0.0*	0.2	0.3	0.3	0.4	0.4	0.01θ	-0.3	-0.5	-0.6	
	0.5	-0.9	-0.7	-0.4	-0.3	-0.2	-0.2	0.0*		-0.5	-0.5	-0.6	
		-0.18	-0.18	0.0*	0.2	0.2	0.3	0.4	0.01θ				
	≥ 10	-1.3**	-1.0	-0.7	-0.5	-0.3	-0.2	0.0*		-0.7	-0.6	-0.6	
		-0.18	-0.18	-0.18	0.0*	0.2	0.2	0.3	0.01θ				
Normal to Ridge for $\theta < 10^\circ$ and Parallel to ridge for all θ	≤ 0.5	Horizontal distance from windward edge			C_p		*Value is provided for interpolation purposes **Value can be reduced linearly with area over which it is applicable as follows						
		0 to $h/2$			-0.9, -0.18								
		$h/2$ to h			-0.9, -0.18								
		h to $2h$			-0.5, -0.18								
		$> 2h$			-0.3, -0.18								
	≥ 1.0	0 to $h/2$			-1.3**, -0.18		Area (m^2)		Reduction Factor				
							$\leq 9.3 m^2$		1.0				
							23.2 m^2		0.9				
		$> h/2$			-0.7, -0.18		$\geq 92.9 m^2$		0.8				

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of L/B , h/L and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 207B.9.4.
6. Refer to Figure 207B.4-2 for domes and Figure 207B.4-3 for arched roofs.
7. Notation:

B = horizontal dimension of building, m, measured normal to wind direction.

L = horizontal dimension of building, m, measured parallel to wind direction.

h = mean roof height in meters, except that eave height shall be used for $\theta \leq 10^\circ$

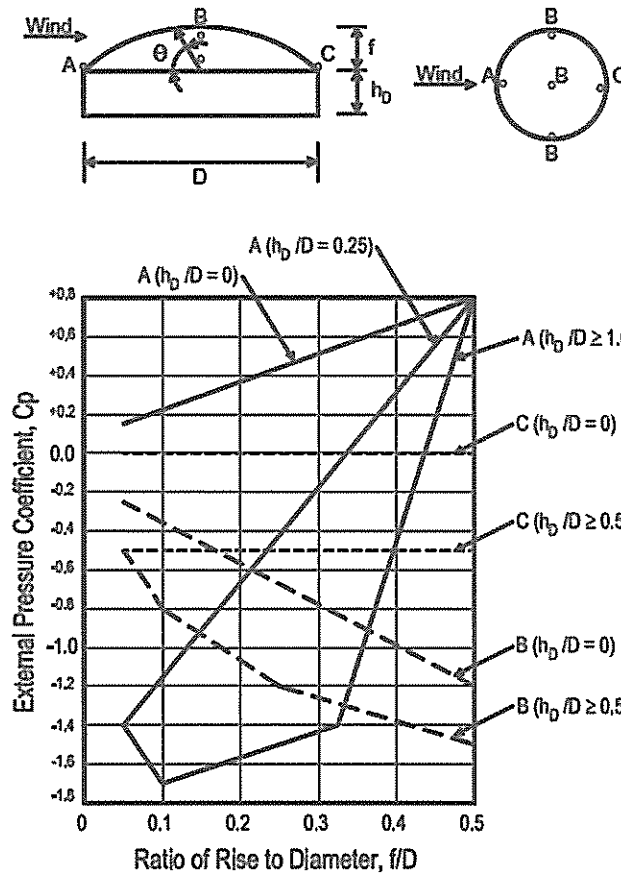
z = height above ground, m

G = gust effect factor.

q_z, q_h = velocity pressure, (N/m^2), evaluated at respective height.

θ = Angle of plane of roof from horizontal, $^\circ$

8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
10. For roof slopes greater than 80° , use $C_p = 0.8$



External Pressure Coefficients for Domes with a Circular Base
(Adapted from Eurocode, 1995)

Notes:

- Two load cases shall be considered:
 - Case A:** C_p values between A and B and between B and C shall be determined by linear interpolation along arcs on the dome parallel to the wind direction;
 - Case B:** C_p shall be the constant value of A for $\theta \leq 25^\circ$, and shall be determined by linear interpolation from 25° to B and from B to C.
- Values denote C_p to be used with $q_{(h_D+f)}$ where $h_D + f$ is the height at the top of the dome.
- Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
- C_p is constant on the dome surface for arcs of circles perpendicular to the wind direction; for example, the arc passing through B-B-B and all arcs parallel to B-B-B.
- For values of h_D/D between those listed on the graph curves, linear interpolation shall be permitted.
- $\theta = 0^\circ$ on dome spring line, $\theta = 90^\circ$ at dome center top point. f is measured from spring line to top.
- The total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
- For f/D values less than 0.05, use Figure 207B.4-1.

Figure 207B.4-2

External Pressure Coefficients, C_p , Domed Roofs Enclosed, Partially Enclosed Buildings

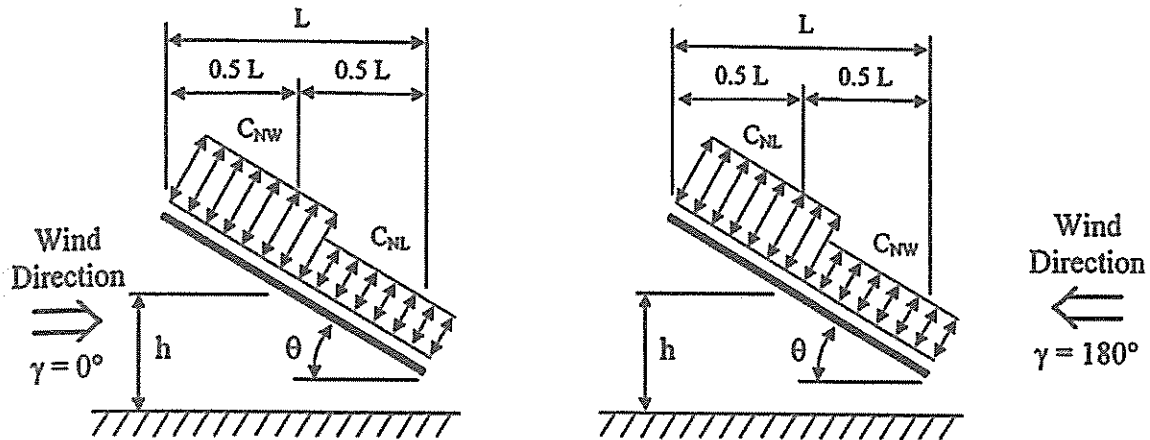
Conditions	Rise-to-span ratio, r	C_p		
		Windward quarter	Center half	Leeward quarter
Roof on elevated structure	$0 < r < 0.2$	-0.9	$-0.7 - r$	-0.5
	$0.2 \leq r < 0.3$ *	$1.5r - 0.3$	$-0.7 - r$	-0.5
	$0.3 \leq r \leq 0.6$	$2.75r - 0.7$	$-0.7 - r$	-0.5
Roof springing from ground level	$0 < r \leq 0.6$	$1.4r$	$-0.7 - r$	-0.5

*When the rise-to-span ratio is $0.2 \leq r \leq 0.3$, alternate coefficients given by $6r - 2.1$ shall also be used for the windward quarter.

Notes:

1. Values listed are for the determination of average loads on main wind force resisting systems.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For wind directed parallel to the axis of the arch, use pressure coefficients from Figure 207B.4-1 with wind directed parallel to ridge.
4. For components and cladding: (1) At roof perimeter, use the external pressure coefficients in Figure 207E.4-2A, B and C with θ based on spring-line slope and (2) for remaining roof areas, use external pressure coefficients of this table multiplied by 0.87.

Figure 207B.4-3
External Pressure Coefficients, C_p , Arched Roofs, $0.25 \leq h/L \leq 1.0$
Enclosed, Partially Enclosed Buildings



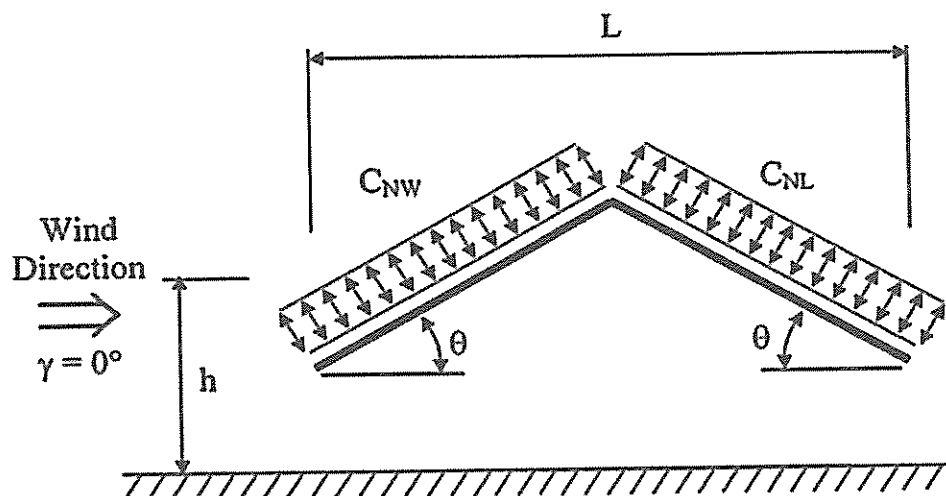
Roof Angle θ	Load Case	Wind Direction, $\gamma = 0^\circ$				Wind Direction, $\gamma = 180^\circ$			
		Clear Wind Flow		Obstructed Wind Flow		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}
0°	A	1.2	0.3	-0.5	-1.2	-1.2	0.3	-0.5	-1.2
	B	-1.1	-0.1	-1.1	-0.6	-1.1	-0.1	-1.1	-0.6
7.5°	A	-0.6	-1	-1	-1.5	0.9	1.5	-0.2	-1.2
	B	-1.4	0	-1.7	-0.8	1.6	0.3	0.8	-0.3
15°	A	-0.9	-1.3	-1.1	-1.5	1.3	1.6	0.4	-1.1
	B	-1.9	0	-2.1	-0.6	1.8	0.6	1.2	-0.3
22.5°	A	-1.5	-1.6	-1.5	-1.7	1.7	1.8	0.5	-1
	B	-2.4	-0.3	-2.3	-0.9	2.2	0.7	1.3	0
30°	A	-1.8	-1.8	-1.5	-1.8	2.1	2.1	0.6	-1
	B	-2.5	-0.5	-2.3	-1.1	2.6	1	1.6	0.1
37.5°	A	-1.8	-1.8	-1.5	-1.8	2.1	2.2	0.7	-0.9
	B	-2.4	-0.6	-2.2	-1.1	2.7	1.1	1.9	0.3
45°	A	-1.6	-1.8	-1.3	-1.8	2.2	2.5	0.8	-0.9
	B	-2.3	-0.7	-1.9	-1.2	2.6	1.4	2.1	0.4

Notes:

- C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.
- Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (> 50% blockage).
- For values of θ between 7.5° and 45° , linear interpolation is permitted. For values of θ less than 7.5° , use load coefficients for 0° .
- Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
- All load cases shown for each roof angle shall be investigated.
- Notation:

- L = horizontal dimension of roof, measured in the along wind direction, m
 h = mean roof height, m
 γ = direction of wind, $^\circ$
 θ = angle of plane of roof from horizontal, $^\circ$

Figure 207B.4-4
 Net Pressure Coefficient, C_N Monoslope Free Roofs $\theta \leq 45^\circ$, $\gamma = 0^\circ, 180^\circ$
 $0.25 \leq h/L \leq 1.0$ Open Buildings



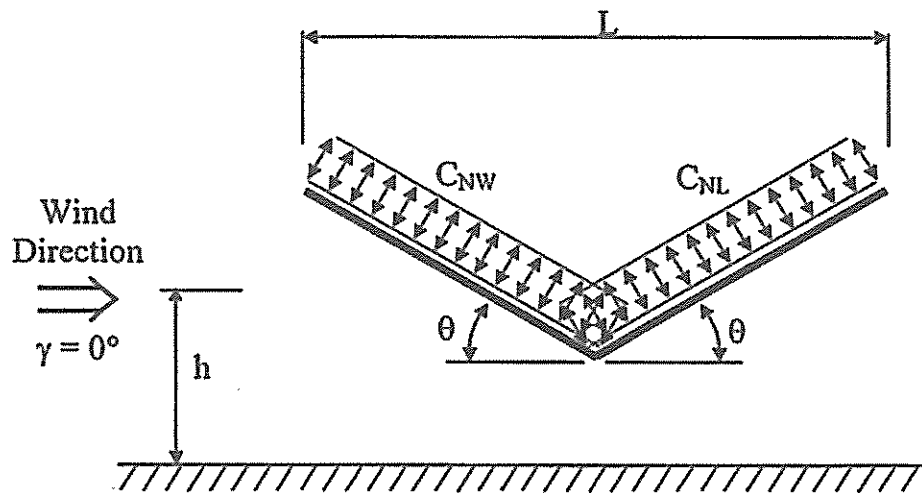
Roof Angle θ	Load Case	Wind Direction, $\gamma = 0^\circ, 180^\circ$			
		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}
7.5°	A	1.1	-0.3	-1.6	-1
	B	0.2	-1.2	-0.9	-1.7
15°	A	1.1	-0.4	-1.2	-1
	B	0.1	-1.1	-0.6	-1.6
22.5°	A	1.1	0.1	-1.2	-1.2
	B	-0.1	-0.8	-0.8	-1.7
30°	A	1.3	0.3	-0.7	-0.7
	B	-0.1	-0.9	-0.2	-1.1
37.5°	A	1.3	0.6	-0.6	-0.6
	B	-0.2	-0.6	-0.3	-0.9
45°	A	1.1	0.9	-0.5	-0.5
	B	-0.3	-0.5	-0.3	-0.7

Notes:

1. C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (> 50% blockage).
3. For values of θ between 7.5° and 45°, linear interpolation is permitted. For values of θ less than 7.5°, use monoslope roof load coefficients.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. All load cases shown for each roof angle shall be investigated.
6. Notation:

L = horizontal dimension of roof, measured in the along wind direction, m
 h = mean roof height, m
 γ = direction of wind, °
 θ = angle of plane of roof from horizontal, °

Figure 207B.4-5
 Net Pressure Coefficient, C_N Pitched Free Roofs $\theta \leq 45^\circ$, $\gamma = 0^\circ, 180^\circ$
 $0.25 \leq h/L \leq 1.0$ Open Buildings



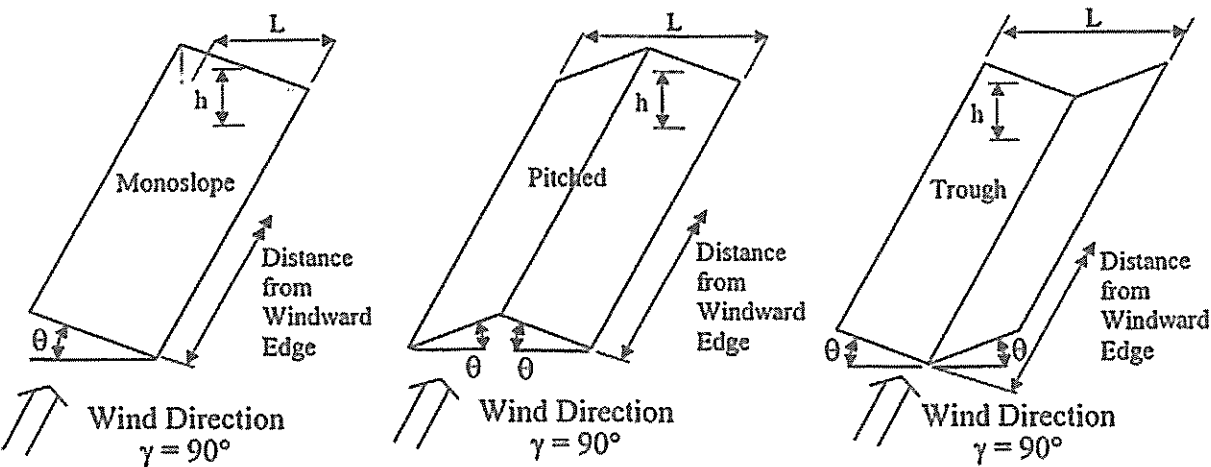
Roof Angle θ	Load Case	Wind Direction, $\gamma = 0^\circ, 180^\circ$			
		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}
7.5°	A	-1.1	0.3	-1.6	-0.5
	B	-0.2	1.2	-0.9	-0.8
15°	A	-1.1	0.4	-1.2	-0.5
	B	0.1	1.1	-0.6	-0.8
22.5°	A	-1.1	-0.1	-1.2	-0.6
	B	-0.1	0.8	-0.8	-0.8
30°	A	-1.3	-0.3	-1.4	-0.4
	B	-0.1	0.9	-0.2	-0.5
37.5°	A	-1.3	-0.6	-1.4	-0.3
	B	0.2	0.6	-0.3	-0.4
45°	A	-1.1	-0.9	-1.2	-0.3
	B	0.3	0.5	-0.3	-0.4

Notes:

1. C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (> 50% blockage).
3. For values of θ between 7.5° and 45°, linear interpolation is permitted. For values of θ less than 7.5°, use monoslope roof load coefficients.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. All load cases shown for each roof angle shall be investigated.
6. Notation:
 - L = horizontal dimension of roof, measured in the along wind direction, m
 - h = mean roof height, m
 - γ = direction of wind, °
 - θ = angle of plane of roof from horizontal, °

Figure 207B.4-6

Net Pressure Coefficient, C_N Troughed Free Roofs $\theta \leq 45^\circ$, $\gamma = 0^\circ, 180^\circ$
 $0.25 \leq h/L \leq 1.0$ Open Buildings



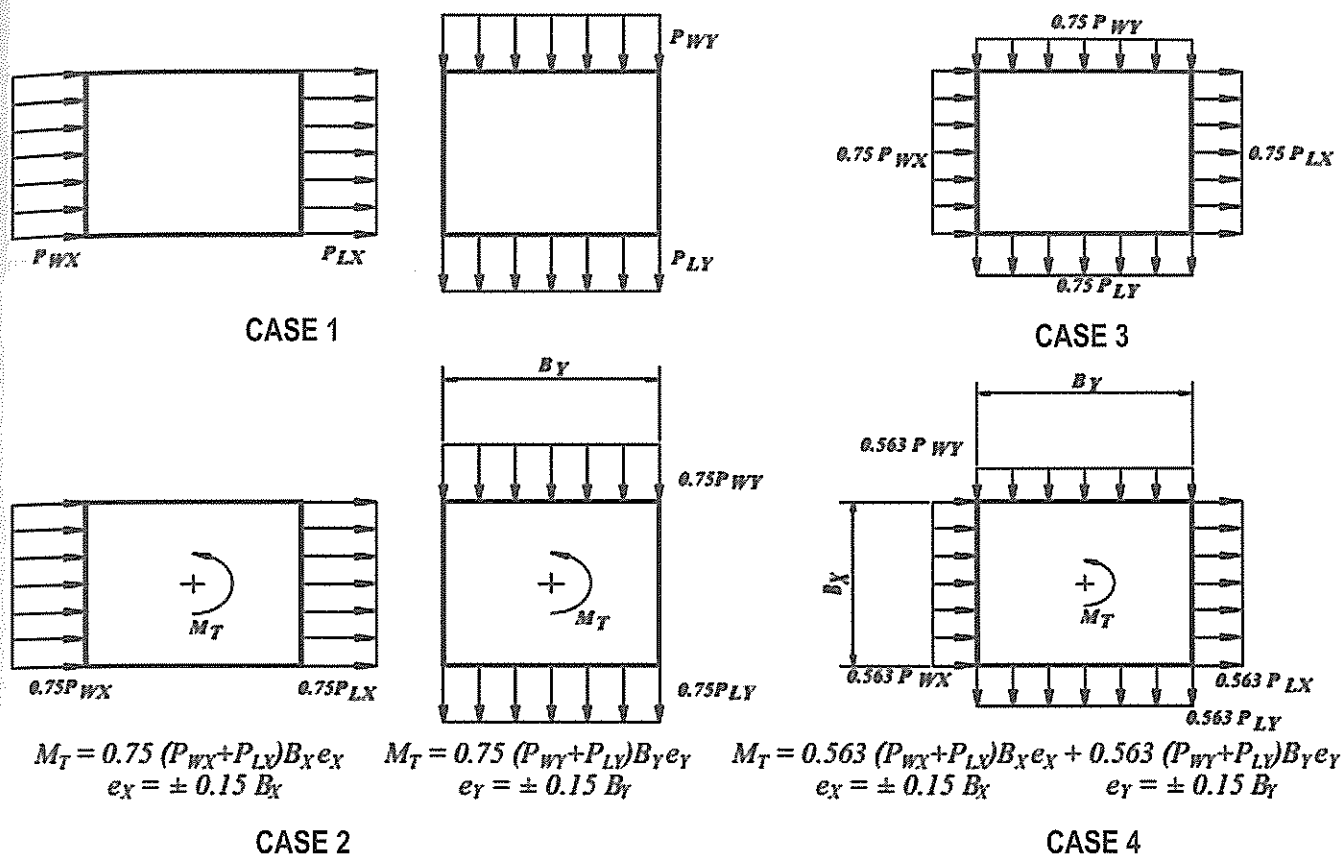
Horizontal Distance from Windward Edge	Roof Angle θ	Load Case	Clear Wind Flow	Obstructed Wind Flow
			C_{NW}	C_{NL}
$\leq h$	All Shapes	A	-0.8	1.2
	$\theta \leq 45^\circ$	B	0.8	0.5
$> h, \leq 2h$	All Shapes	A	-0.6	-0.9
	$\theta \leq 45^\circ$	B	0.5	0.5
$> 2h$	All Shapes	A	-0.3	-0.6
	$\theta \leq 45^\circ$	B	0.3	0.3

Notes:

- 1. CN denotes net pressures (contributions from top and bottom surfaces).
- 2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (> 50% blockage).
- 3. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
- 4. All load cases shown for each roof angle shall be investigated.
- 5. For monoslope roofs with theta less than 5 degrees, C_N values shown apply also for cases where gamma = 0 degrees and 0.05 less than or equal to h/L less than or equal to 0.25. See Figure 207B.4-4 for other h/L values.
- 6. Notation:

- L = horizontal dimension of roof, measured in the along wind direction, m
- h = mean roof height, m. See Figures 207B.4-4, 207B.4-5 or 207B.4-6 for a graphical depiction of this dimension.
- γ = direction of wind, °
- θ = angle of plane of roof from horizontal, °

Figure 207B.4-7
Net Pressure Coefficient, C_N Free Roofs $\theta \leq 45^\circ$, $\gamma = 90^\circ$, 270°
 $0.25 \leq h/L \leq 1.0$ Open Buildings



Case 1: Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2: Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3: Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4: Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 207B.4.1 and 207B.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

P_{WX}, P_{WY}	=	Windward face design pressure acting in the x, y principal axis, respectively.
P_{LX}, P_{LY}	=	Leeward face design pressure acting in the x, y principal axis, respectively.
$e(e_X, e_Y)$	=	Eccentricity for the x, y principal axis of the structure, respectively.
M_T	=	Torsional moment per unit height acting about a vertical axis of the building.

Figure 207B.4-8
Design Wind Load Cases All Heights

Part 2: Enclosed Simple Diaphragm Buildings with $h = 48$ m

This section has been added to ASCE 7-10 to cover the common practical cases of enclosed simple diaphragm buildings up to height $h = 48$ m. Two classes of buildings are covered by this method. Class 1 buildings have $h \leq 18$ m with plan aspect ratios L/B between 0.2 and 5.0. Cases A through F are described in Appendix D, ASCE 7-10 to allow the designer to establish the lines of resistance of the MWFRS in each direction so that the torsional load cases of Figure 207B.4-8 need not be considered. Class 2 buildings have $18\text{ m} < h \leq 48$ m with plan aspect ratios of L/B between 0.5 and 2.0. Cases A through E of Appendix D, ASCE 7-10 are described to allow the designer to establish the lines of resistance of the MWFRS so that the torsional load cases of Figure 207B.4-8 need not be considered.

For the type of buildings covered in this method, the internal building pressure cancels out and need not be considered for the design of the MWFRS. Design net wind pressures for roofs and walls are tabulated directly in Tables 207B.6-1 and 207B.6-2 using the Directional Procedure as described in Part 1. Guidelines for determining the exterior pressures on windward, leeward, and side walls are provided in footnotes to Table 207B.6-1.

The requirements in Class 2 buildings for natural building frequency ($75/h$) and structural damping ($\beta = 1.5\%$ critical) are necessary to ensure that the Gust Effect Factor, G_f , which has been calculated and built into the design procedure, is consistent with the tabulated pressures. The frequency of $75/h$ represents a reasonable lower bound to values found in practice. If calculated frequencies are found to be lower, then consideration should be given to stiffening the building. A structural damping value of 1.5 %, applicable at the ultimate wind speeds as defined in the new wind speed maps, is conservative for most common building types and is consistent with a damping value of 1% for the ultimate wind speeds divided by 1.6 as contained in the NSCP 2010 (ASCE 7-05) wind speed map. Because Class 1 buildings are limited to $h \leq 18$ m, the building can be assumed to be rigid as defined in the glossary, and the Gust Effect Factor can be assumed to be 0.85. For this class of buildings frequency and damping need not be considered.

207B.5 General Requirements

207B.5.1 Design Procedure

The procedure specified herein applies to the determination of MWFRS wind loads of enclosed simple diaphragm buildings, as defined in Section 207A.2, with a mean roof height $h \leq 48$ m. The steps required for the determination of MWFRS wind loads on enclosed simple diaphragm buildings are shown in Table 207B.5-1.

User Note:

Part 2 of Section 207B is a simplified method for determining the wind pressures for the MWFRS of enclosed, simple diaphragm buildings whose height h is ≤ 4 m. The wind pressures are obtained directly from a table. The building may be of any general plan shape and roof geometry that matches the specified figures. This method is a simplification of the traditional "all heights" method (Directional Procedure) contained in Part 1 of Section 207B.

Table 207B.5-1
Steps to Determine MWFRS Wind
Loads Enclosed Simple Diaphragm Buildings
($h \leq 48$ m)

Step 1:	Determine risk category of building or other structure, see Table 103-1
Step 2:	Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C
Step 3:	Determine wind load parameters: ➤ Wind directionality factor, K_d, see Section 207A.6 and Table 207A.6-1 ➤ Exposure category B, C or D, see Section 207A.7 ➤ Topographic factor, K_{zt}, see Section 207A.8 and Figure 207A.8-1 ➤ Enclosure classification, see Section 207A.10
Step 4:	Enter table to determine net pressures on walls at top and base of building respectively, p_h , p_o , Table 207B.6-1
Step 5:	Enter table to determine net roof pressures, p_z , Table 207B.6-2
Step 6:	Determine topographic factor, K_{zt} , and apply factor to wall and roof pressures (if applicable), see Section 207A.8
Step 7:	Apply loads to walls and roofs simultaneously.

207B.5.2 Conditions

In addition to the requirements in Section 207B.1.2, a building whose design wind loads are determined in accordance with this section shall meet all of the following conditions for either a Class 1 or Class 2 building (see Figure 207B.5-1):

Class 1 Buildings:

1. The building shall be an enclosed simple diaphragm building as defined in Section 207A.2.
2. The building shall have a mean roof height $h \leq 18$ m.
3. The ratio of L/B shall not be less than 0.2 nor more than 5.0 ($0.2 \leq L/B \leq 5.0$).

4. The topographic effect factor $K_{zt} = 1.0$ or the wind pressures determined from this section shall be multiplied by K_{zt} at each height z as determined from Section 207A.8. It shall be permitted to use one value of K_{zt} for the building calculated at $0.33h$. Alternatively it shall be permitted to enter the pressure table with a wind velocity equal to $V\sqrt{K_{zt}}$ where K_{zt} is determined at a height of $0.33h$.

Class 2 Buildings:

1. The building shall be an enclosed simple diaphragm building as defined in Section 207A.2.
2. The building shall have a mean roof height 18 m ($18 \text{ m} < h \leq 48 \text{ m}$).
3. The ratio of L/B shall not be less than 0.5 nor more than 2.0 ($0.5 \leq L/B \leq 2.0$).
4. The fundamental natural frequency (Hertz) of the building shall not be less $75/h$ where h is in meters.
5. The topographic effect factor $K_{zt} = 1.0$ or the wind pressures determined from this section shall be multiplied by K_{zt} at each height z as determined from Section 207A.8. It shall be permitted to use one value of K_{zt} for the building calculated at $0.33h$. Alternatively it shall be permitted to enter the pressure table with a wind velocity equal to $V\sqrt{K_{zt}}$ where K_{zt} is determined at a height of $0.33h$.

207B.5.3 Wind Load Parameters Specified in Section 207A

Refer to Section 207A for determination of Basic Wind Speed V (Section 207A.5) and exposure category (Section 207A.7) and topographic factor K_{zt} (Section 207A.8).

207B.5.4 Diaphragm Flexibility

The design procedure specified herein applies to buildings having either rigid or flexible diaphragms. The structural analysis shall consider the relative stiffness of diaphragms and the vertical elements of the MWFRS.

Diaphragms constructed of wood panels can be idealized as flexible. Diaphragms constructed of untopped metal decks, concrete filled metal decks, and concrete slabs, each having a span-to-depth ratio of 2 or less, are permitted to be idealized as rigid for consideration of wind loading.

207B.6 Wind Loads—Main Wind Force-Resisting System

207B.6.1 Wall and Roof Surfaces—Class 1 and 2 Buildings

Net wind pressures for the walls and roof surfaces shall be determined from Tables 207B.6-1 and 207B.6-2, respectively, for the applicable exposure category as determined by Section 207A.7.

For Class 1 building with L/B values less than 0.5, use wind pressures tabulated for $L/B = 0.5$. For Class 1 building with L/B values greater than 2.0, use wind pressures tabulated for $L/B = 2.0$.

Net wall pressures shall be applied to the projected area of the building walls in the direction of the wind, and exterior side wall pressures shall be applied to the projected area of the building walls normal to the direction of the wind acting outward according to Note 3 of Table 207B.6-1, simultaneously with the roof pressures from Table 207B.6-2 as shown in Figure 207B.6-1.

Where two load cases are shown in the table of roof pressures, the effects of each load case shall be investigated separately. The MWFRS in each direction shall be designed for the wind load cases as defined in Figure 207B.4-8.

Exception:

The torsional load cases in Figure 207B.4-8 (Case 2 and Case 4) need not be considered for buildings which meet the requirements of Appendix D, ASCE 7-10.

Commentary:

Wall and roof net pressures are shown in Tables 207B.6-1 and 207B.6-2 and are calculated using the external pressure coefficients in Figure 207B.4-1. Along wind net wall pressures are applied to the projected area of the building walls in the direction of the wind, and exterior sidewall pressures are applied to the projected area of the building walls normal to the direction of the wind acting outward, simultaneously with the roof pressures from Table 207B.6-2. Distribution of the net wall pressures between windward and leeward wall surfaces is defined in Note 4 of Table 207B.6-1. The magnitude of exterior sidewall pressure is determined from Note 2 of Table 207B.6-1. It is to be noted that all tabulated pressures are defined without consideration of internal pressures because internal pressures cancel out when considering the net effect on the MWFRS of simple diaphragm buildings. Where the net wind pressure on any individual

wall surface is required, internal pressure must be included as defined in Part 1 of Section 207B.

The distribution of wall pressures between windward and leeward wall surfaces is useful for the design of floor and roof diaphragm elements like drag strut collector beams, as well as for MWFRS wall elements. The values defined in Note 4 of Table 207B.6-1 are obtained as follows: The external pressure coefficient for all windward walls is $C_p = 0.8$ for all L/B values. The leeward wall C_p value is (-0.5) for L/B values from 0.5 to 1.0 and is (-0.3) for $L/B = 2.0$. Noting that the leeward wall pressure is constant for the full height of the building, the leeward wall pressure can be calculated as a percentage of the p_h value in the table. The percentage is $0.5/(0.8 + 0.5) \times 100 = 38\%$ for $L/B = 0.5$ to 1.0. The percentage is $0.3/(0.8 + 0.3) \times 100 = 27\%$ for $L/B = 2.0$. Interpolation between these two percentages can be used for L/B ratios between 1.0 and 2.0. The windward wall pressure is then calculated as the difference between the total net pressure from the table using the p_h and p_o values and the constant leeward wall pressure.

Sidewall pressures can be calculated in a similar manner to the windward and leeward wall pressures by taking a percentage of the net wall pressures. The C_p value for sidewalls is (-0.7) . Thus, for $L/B = 0.5$ to 1.0, the percentage is $0.7/(0.8 + 0.5) \times 100 = 54\%$. For $L/B = 2.0$, the percentage is $0.7/(0.8 + 0.3) \times 100 = 64\%$. Note that the sidewall pressures are constant up the full height of the building.

The pressures tabulated for this method are based on simplifying conservative assumptions made to the different pressure coefficient (GC_p) cases tabulated in Figure 207B.4-1, which is the basis for the traditional all heights building procedure (defined as the Directional Procedure in this code) that has been a part of the standard since 1972. The external pressure coefficients C_p for roofs have been multiplied by 0.85, a reasonable gust effect factor for most common roof framing, and then combined with an internal pressure coefficient for enclosed buildings (plus or minus 0.18) to obtain a net pressure coefficient to serve as the basis for pressure calculation. The linear wall pressure diagram has been conceived so that the applied pressures from the table produce the same overturning moment as the more exact pressures from Part 1 of Section 207B. For determination of the wall pressures tabulated, the actual gust effect factor has been calculated from Equation 207A.9-10 based on building height, wind speed, exposure, frequency, and the assumed damping value.

207B.6.2 Parapets

The effect of horizontal wind loads applied to all vertical surfaces of roof parapets for the design of the MWFRS shall be based on the application of an additional net horizontal wind pressure applied to the projected area of the parapet surface equal to 2.25 times the wall pressures tabulated in Table 207B.6-1 for $L/B = 1.0$. The net pressure specified accounts for both the windward and leeward parapet loading on both the windward and leeward building surface. The parapet pressure shall be applied simultaneously with the specified wall and roof pressures shown in the table as shown in Figure 207B.6-2. The height h used to enter Table 207B.6-1 to determine the parapet pressure shall be the height to the top of the parapet as shown in Figure 207B.6-2 (use $h = h_p$).

Commentary:

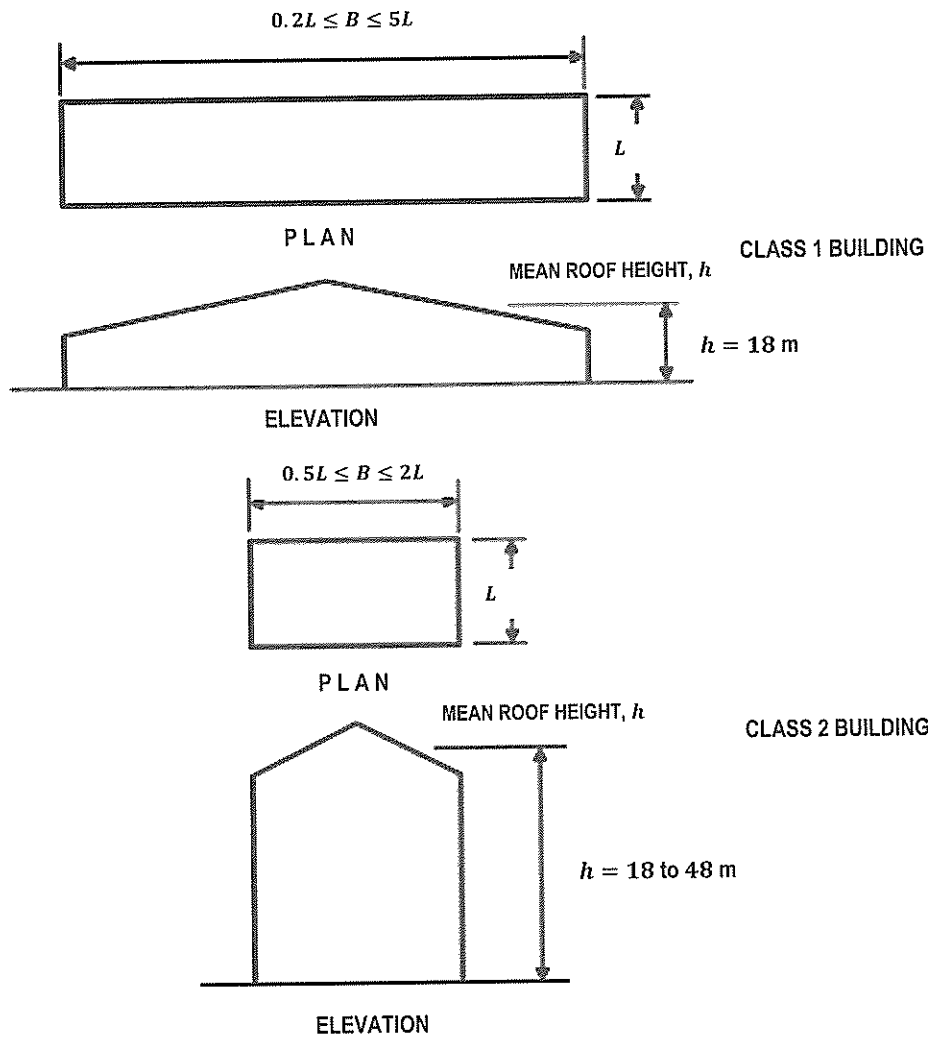
The effect of parapet loading on the MWFRS is specified in Section 207B.4.5 of Part 1. The net pressure coefficient for the windward parapet is +1.5 and for the leeward parapet is -1.0. The combined effect of both produces a net coefficient of +2.5 applied to the windward surface to account for the cumulative effect on the MWFRS in a simple diaphragm building. This pressure coefficient compares to a net pressure coefficient of $1.3G_f$ for the tabulated horizontal wall pressure p_h at the top of the building. Assuming a lower-bound gust factor $G_f = 0.85$, the ratio of the parapet pressure to the wall pressure is $2.5/(0.85 \times 1.3) = 2.25$. Thus, a value of 2.25 is assumed as a reasonable constant to apply to the tabulated wall pressure p_h to account for the additional parapet loading on the MWFRS.

207B.6.3 Roof Overhangs

The effect of vertical wind loads on any roof overhangs shall be based on the application of a positive wind pressure on the underside of the windward overhang equal to 75% of the roof edge pressure from Table 207B.6-2 for Zone 1 or Zone 3 as applicable. This pressure shall be applied to the windward roof overhang only and shall be applied simultaneously with other tabulated wall and roof pressures as shown in Figure 207B.6-3.

Commentary:

The effect of vertical wind loading on a windward roof overhang is specified in Section 207B.4.4 of Part 1. A positive pressure coefficient of +0.8 is specified. This compares to a net pressure coefficient tabulated for the windward edge zone 3 of -1.06 (derived from $0.85 \times -1.3 \times 0.8 = -0.18$). The 0.85 factor represents the gust factor G , the 0.8 multiplier accounts for the effective wind area reduction to the 1.3 value of C_p specified in Figure 207B.4-1 of Part 1, and the -0.18 is the internal pressure contribution. The ratio of coefficients is $0.8/1.06 = 0.755$. Thus, a multiplier of 0.75 on the tabulated pressure for zone 3 in Table 207B.6-2 is specified.



Note: Roof form may be flat, gable, mansard or hip

Figure 207B.5-1
 Building Geometry Requirements Building Class, $h \leq 48$ m
 Enclosed Simple Diaphragm Buildings

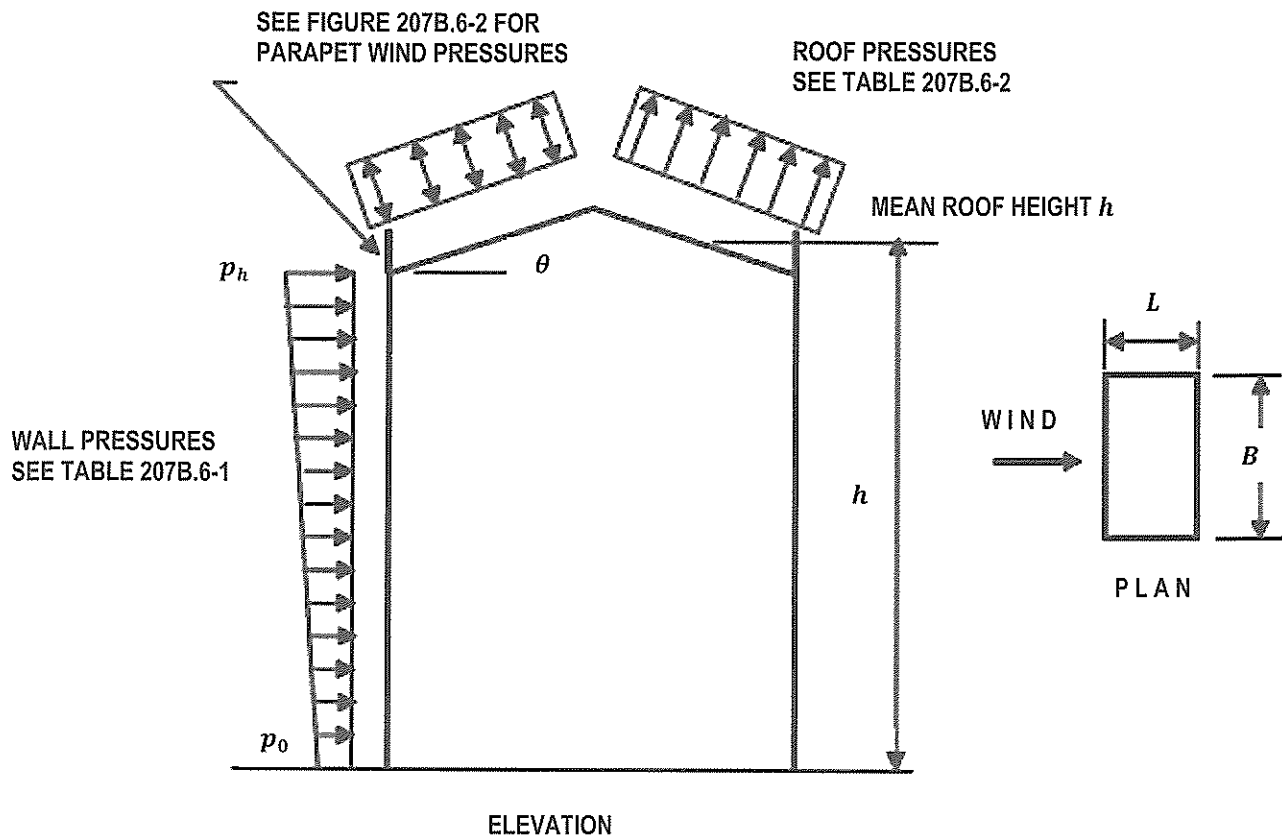


Figure 207B.6-1
Application of Wind Pressures – Walls and Roof, $h \leq 48$ m
Enclosed Simple Diaphragm Buildings

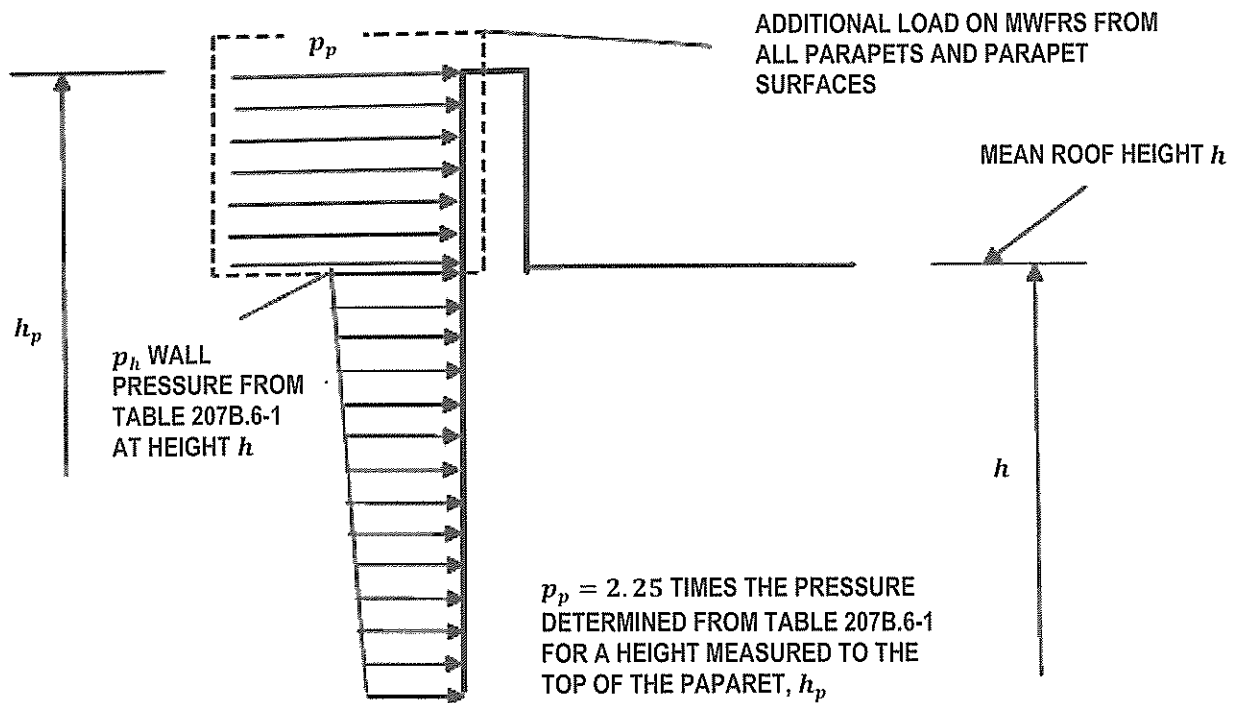


Figure 207B.6-2
Application of Parapet Wind Loads Parapet Wind Loads, $h \leq 48$ m
Enclosed Simple Diaphragm Buildings

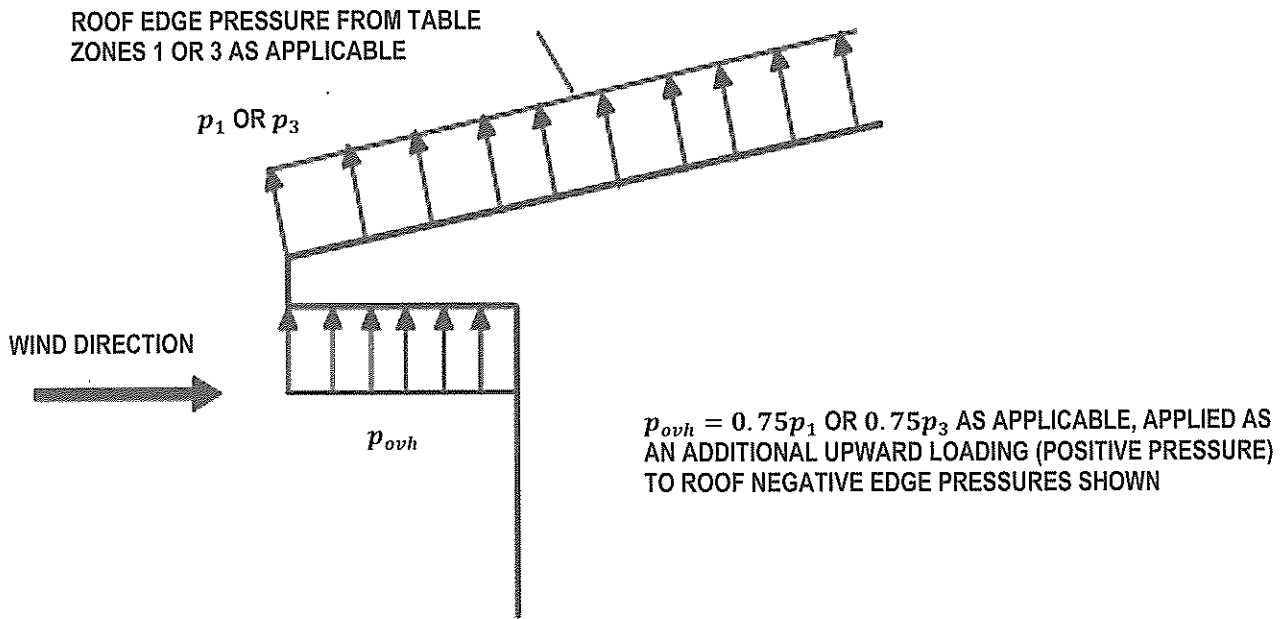
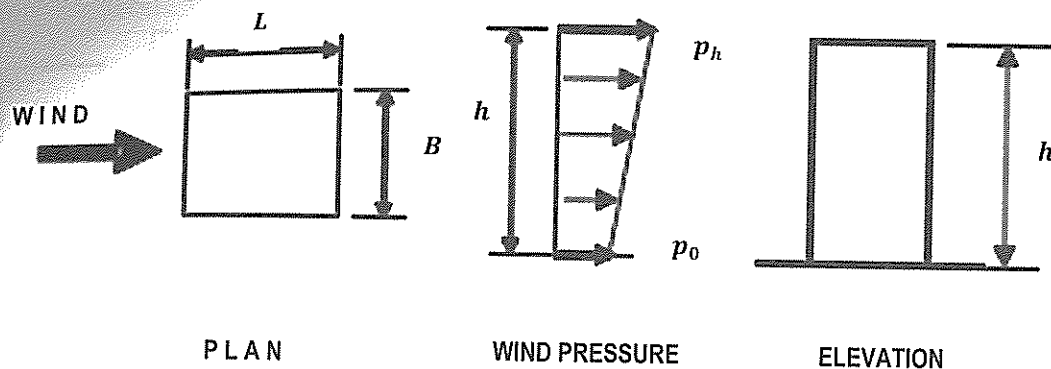


Figure 207B.6-3
Application of Roof Overhang Wind Loads, $h \leq 48$ m
Enclosed Simple Diaphragm Buildings



Notes to Wall Pressure Table 207B.6-1:

1. From table for each Exposure (B, C or D), V , L/B and h , determine p_h (top number) and p_o (bottom number) horizontal along-wind net wall pressures.
2. Side wall external pressures shall be uniform over the wall surface acting outward and shall be taken as 54% of the tabulated p_h pressure for $0.2 \leq L/B \leq 1.0$ and 64% of the tabulated p_h pressure for $2.0 \leq L/B \leq 5.0$. Linear interpolation shall apply for $1.0 < L/B < 2.0$. Side wall external pressures do not include effect of internal pressure.
3. Apply along-wind net wall pressures as shown above to the projected area of the building walls in the direction of the wind and apply external side wall pressures to the projected area of the building walls normal to the direction wind, simultaneously with the roof pressures from Table 207B.6-2.
4. Distribution of tabulated net wall pressures between windward and leeward wall faces shall be based on the linear distribution of total net pressure with building height as shown above and the leeward external wall pressures assumed uniformly distributed over the leeward wall surface acting outward at 38% of p_h for $0.2 \leq L/B \leq 1.0$ and 27% of p_h for $2.0 \leq L/B \leq 5.0$. Linear interpolation shall be used for $1.0 < L/B < 2.0$. The remaining net pressure shall be applied to the windward walls as an external wall pressure acting towards the wall surface. Windward and leeward wall pressures so determined do not include effect of internal pressure.
5. Interpolation between values of V , h and L/B is permitted.

Notation:

L	=	building plan dimension parallel to wind direction, m
B	=	building plan dimension perpendicular to wind direction, m
h	=	mean roof height, m
p_h, p_o	=	along-wind net wall pressure at top and base of building respectively, Pa

Figure 207B.6-1
Application of Wall Pressures Wind Pressures - Walls, $h \leq 48$ m
Enclosed Simple Diaphragm Buildings

Table 207B.6-1
MWFRS – Part 2: Wind Loads – Walls (kN/m²)
Exposure B

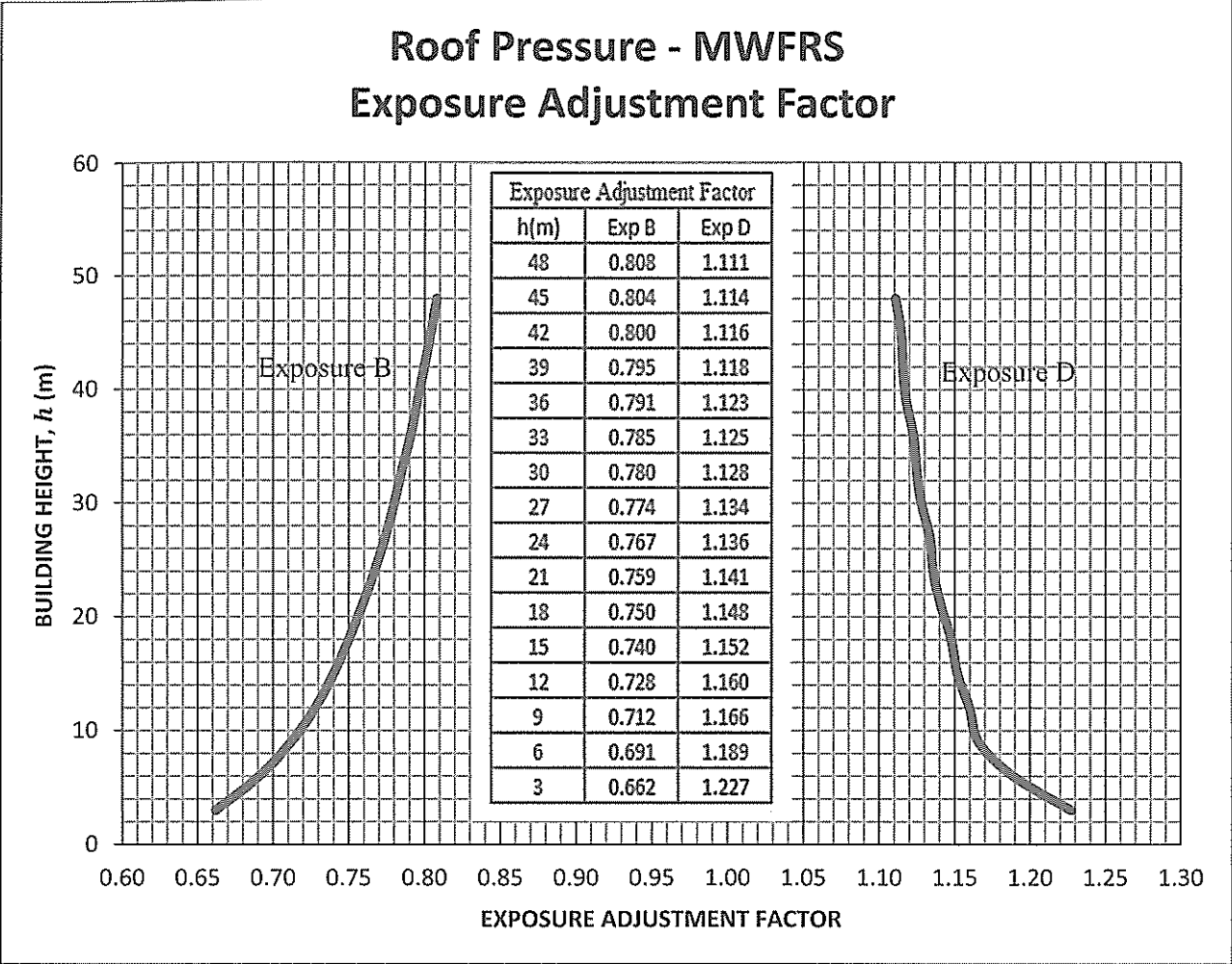
V (kph)	150			200			250			300			350		
h (m), L/B	0.5	1	2	0.5	1	2	0.5	1	2	0.5	1	2	0.5	1	2
48	1.24	1.22	1.09	2.39	2.36	2.15	4.03	3.96	3.63	6.25	6.11	5.61	9.14	8.90	8.17
	0.83	0.82	0.67	1.61	1.59	1.33	2.72	2.67	2.24	4.22	4.12	3.47	6.17	6.00	5.09
45	1.20	1.19	1.06	2.31	2.29	2.08	3.88	3.82	3.50	6.00	5.88	5.40	8.76	8.57	7.86
	0.82	0.81	0.66	1.58	1.56	1.30	2.65	2.61	2.19	4.09	4.01	3.39	5.96	5.81	4.96
42	1.16	1.15	1.03	2.23	2.21	2.00	3.73	3.68	3.37	5.75	5.64	5.19	8.38	8.17	7.51
	0.79	0.80	0.65	1.54	1.53	1.27	2.57	2.54	2.14	3.97	3.89	3.30	5.83	5.64	4.79
39	1.13	1.12	1.00	2.14	2.13	1.93	3.57	3.53	3.23	5.49	5.40	4.97	7.97	7.82	7.22
	0.78	0.79	0.64	1.50	1.49	1.25	2.50	2.47	2.09	3.84	3.78	3.21	5.58	5.47	4.66
36	1.08	1.09	0.96	2.06	2.05	1.85	3.42	3.38	3.09	5.23	5.15	4.74	7.56	7.43	6.86
	0.77	0.77	0.64	1.46	1.45	1.22	2.42	2.40	2.03	3.71	3.66	3.12	5.39	5.27	4.54
33	1.04	1.05	0.93	1.98	1.97	1.77	3.26	3.23	2.94	4.97	4.90	4.51	7.20	7.05	6.55
	0.76	0.76	0.63	1.43	1.42	1.19	2.35	2.33	1.98	3.58	3.54	3.03	5.18	5.10	4.37
30	1.01	1.00	0.89	1.90	1.88	1.69	3.10	3.08	2.80	4.70	4.65	4.27	6.79	6.64	6.15
	0.73	0.74	0.61	1.39	1.38	1.16	2.27	2.26	1.92	3.45	3.41	2.93	5.01	4.86	4.23
27	0.97	0.96	0.86	1.85	1.80	1.60	2.94	2.92	2.65	4.43	4.39	4.02	6.51	6.28	5.72
	0.73	0.72	0.60	1.35	1.34	1.13	2.20	2.19	1.86	3.32	3.28	2.83	4.75	4.62	4.08
24	0.93	0.92	0.81	1.71	1.71	1.52	2.77	2.76	2.49	4.16	4.13	3.77	5.93	5.88	5.41
	0.71	0.72	0.60	1.31	1.31	1.10	2.12	2.11	1.80	3.18	3.16	2.72	4.53	4.50	3.88
21	0.88	0.89	0.77	1.62	1.61	1.43	2.60	2.59	2.33	3.88	3.86	3.50	5.52	5.45	4.97
	0.69	0.69	0.57	1.27	1.27	1.07	2.04	2.04	1.74	3.05	3.03	2.61	4.35	4.27	3.71
18	0.83	0.85	0.73	1.52	1.52	1.34	2.43	2.42	2.16	3.60	3.58	3.23	5.07	5.03	4.59
	0.68	0.68	0.55	1.23	1.23	1.03	1.97	1.96	1.67	2.91	2.90	2.50	4.06	4.08	3.55
15	0.78	0.78	0.69	1.41	1.41	1.24	2.25	2.25	1.99	3.31	3.30	2.95	4.60	4.56	4.13
	0.65	0.65	0.56	1.19	1.19	1.00	1.89	1.89	1.61	2.78	2.77	2.39	3.89	3.85	3.34
12	0.74	0.74	0.63	1.31	1.31	1.14	2.07	2.07	1.82	3.03	3.03	2.69	4.20	4.20	3.77
	0.65	0.63	0.52	1.15	1.15	0.97	1.82	1.82	1.55	2.66	2.66	2.29	3.67	3.69	3.22
9	0.67	0.67	0.57	1.19	1.19	1.03	1.88	1.88	1.64	2.74	2.74	2.40	3.77	3.77	3.31
	0.62	0.62	0.53	1.10	1.10	0.94	1.74	1.74	1.49	2.53	2.53	2.19	3.46	3.46	3.05
6	0.59	0.59	0.51	1.07	1.07	0.93	1.68	1.68	1.46	2.43	2.43	2.12	3.33	3.33	2.93
	0.59	0.59	0.51	1.05	1.05	0.91	1.65	1.65	1.43	2.39	2.39	2.08	3.27	3.27	2.87
3	0.38	0.38	0.33	0.67	0.67	0.58	1.05	1.05	0.91	1.51	1.51	1.33	2.04	2.04	1.85
	0.38	0.38	0.33	0.67	0.67	0.58	1.05	1.05	0.91	1.51	1.51	1.33	2.04	2.04	1.85

Table 207B.6-1
MWFRS – Part 2: Wind Loads – Walls (kN/m²)
Exposure C

V (kph)	150			200			250			300			350		
h (m), L/B	0.5	1	2	0.5	1	2	0.5	1	2	0.5	1	2	0.5	1	2
48	1.58	1.58	1.42	3.10	3.06	2.75	5.21	5.11	4.62	8.04	7.85	7.10	11.72	11.40	7.10
	1.16	1.16	0.97	2.28	2.25	1.89	3.83	3.76	3.19	5.92	5.77	4.89	8.66	8.36	4.89
45	1.56	1.55	1.38	3.01	2.97	2.68	5.06	5.04	4.49	7.79	7.61	6.89	11.28	10.53	6.89
	0.68	1.16	0.96	2.24	2.21	1.87	3.75	3.74	3.13	5.78	5.65	4.81	8.90	7.84	4.81
42	1.56	1.51	1.33	2.92	2.89	2.60	4.90	4.87	4.36	7.53	7.37	6.68	10.84	10.31	6.68
	1.18	1.13	0.94	2.19	2.17	1.85	3.67	3.65	3.07	5.65	5.53	4.71	8.16	7.77	4.71
39	1.48	1.48	1.29	2.83	2.81	2.52	4.73	4.66	4.22	7.26	7.12	6.46	10.5	10.28	6.46
	1.13	1.12	0.94	2.15	2.13	1.81	3.59	3.53	3.01	5.50	5.39	4.62	7.93	7.78	4.62
36	1.43	1.43	1.27	2.74	2.72	2.44	4.56	4.50	4.07	6.98	6.85	6.23	10.09	9.85	6.23
	1.10	1.10	0.92	2.10	2.09	1.83	3.50	3.45	2.95	5.35	5.25	4.51	7.70	7.56	4.51
33	1.39	1.39	1.23	2.65	2.63	2.36	4.39	4.33	3.92	6.69	6.59	5.98	9.63	9.51	5.98
	1.08	1.09	0.90	2.06	2.04	1.97	3.41	3.37	2.89	5.20	5.11	4.41	7.50	7.29	4.41
30	1.37	1.36	1.20	2.55	2.44	2.27	4.21	4.17	3.76	6.40	6.31	5.73	9.17	8.62	5.73
	1.08	1.06	0.89	2.01	2.00	1.70	3.31	3.28	2.82	5.04	4.97	4.30	7.26	7.14	4.30
27	1.32	1.31	1.16	2.45	1.90	2.18	4.02	3.99	3.60	6.09	6.02	5.47	8.72	6.43	5.47
	1.06	1.06	0.87	1.96	1.95	1.67	3.22	3.19	2.75	4.87	4.82	4.18	6.94	6.88	4.18
24	1.26	1.27	1.11	2.35	2.34	2.08	3.83	3.81	3.42	5.78	5.72	5.19	8.28	8.11	5.19
	1.04	1.07	0.87	1.92	1.91	1.63	3.12	3.10	2.67	4.70	4.66	4.05	6.72	6.61	4.05
21	1.23	1.21	1.06	2.24	2.24	1.98	3.63	3.62	3.24	5.45	5.41	4.89	7.75	7.67	4.89
	1.02	1.26	0.84	1.90	1.86	1.59	3.02	2.99	2.59	4.53	4.34	3.92	6.58	5.60	3.92
18	1.16	1.16	1.01	2.13	2.13	1.88	3.43	3.42	3.05	5.12	5.08	4.58	7.26	7.16	4.58
	0.99	1.00	0.83	2.07	1.81	1.55	2.92	2.75	2.51	4.36	3.05	3.77	7.21	1.94	3.77
15	1.10	1.07	0.99	2.01	2.01	1.76	3.22	3.21	2.85	4.77	4.75	4.25	6.70	6.71	4.25
	0.98	0.96	0.81	1.76	1.76	1.50	2.82	2.81	2.43	4.17	4.16	3.63	5.82	5.86	3.63
12	1.05	0.56	0.91	1.89	1.89	1.64	3.01	2.99	2.63	4.41	4.39	3.90	6.09	6.62	3.90
	0.94	0.94	0.80	1.71	1.71	1.46	2.71	2.71	2.34	3.99	3.98	3.47	5.60	5.56	3.47
9	0.98	0.97	0.83	1.75	1.75	1.51	2.87	2.76	2.41	4.02	4.02	3.53	4.88	5.55	3.53
	0.92	0.92	0.78	1.65	1.65	1.41	2.60	2.60	2.25	3.80	3.79	3.30	5.28	5.24	3.30
6	1.01	0.90	0.78	1.60	1.60	1.38	2.55	2.53	2.40	3.65	3.65	3.18	4.69	4.92	3.18
	0.87	0.87	0.76	1.58	1.58	1.36	2.49	2.48	2.37	3.60	3.60	3.13	4.91	4.97	3.13
3	0.56	0.56	0.49	1.01	1.01	0.88	1.59	1.59	1.38	2.30	2.30	2.00	3.14	3.14	2.00
	0.56	0.56	0.49	1.01	1.01	0.88	1.59	1.59	1.38	2.30	2.30	2.00	3.14	3.14	2.00

Table 207B.6-1
 MWFRS – Part 2: Wind Loads – Walls (kN/m²)
 Exposure D

V (kph)	150			200			250			300			350		
h (m), L/B	0.5	1	2	0.5	1	2	0.5	1	2	0.5	1	2	0.5	1	2
48	1.80	1.80	1.59	3.50	3.45	3.08	5.87	5.75	5.15	9.00	8.79	7.86	12.98	12.66	11.27
	1.39	1.36	1.16	2.70	2.66	2.25	4.52	4.43	3.76	6.93	6.77	5.74	10.01	9.78	8.24
45	1.77	1.76	1.56	3.42	3.37	3.01	5.72	5.61	5.02	8.76	8.56	7.67	12.63	12.30	11.04
	1.37	1.37	1.15	2.66	2.62	2.22	4.44	4.36	3.70	6.80	6.65	5.65	9.83	9.55	8.13
42	1.74	1.73	1.53	3.33	3.29	2.94	5.56	5.46	4.89	8.51	8.33	7.46	12.26	11.99	10.73
	1.36	1.36	1.13	2.61	2.58	2.19	4.36	4.28	3.65	6.66	6.52	5.56	9.56	9.36	7.97
39	1.69	1.69	1.49	3.24	3.21	2.86	5.40	5.31	4.76	8.24	8.08	7.25	11.83	11.61	10.39
	1.34	1.29	1.12	2.57	2.54	2.16	4.27	4.20	3.59	6.52	6.39	5.46	9.4	9.23	7.82
36	1.64	1.65	1.45	3.15	3.12	2.78	5.23	5.15	4.62	7.97	7.82	7.03	11.46	11.21	10.07
	1.31	1.17	1.13	2.52	2.49	2.12	4.18	4.12	3.52	6.37	6.25	5.36	9.17	9.07	7.67
33	1.87	1.61	1.41	3.06	3.03	2.70	5.05	4.99	4.47	7.69	7.55	6.80	10.83	10.77	9.77
	1.31	1.29	1.16	2.47	2.45	2.08	4.08	4.03	3.46	6.21	6.10	5.26	8.93	8.73	7.44
30	2.78	1.57	1.38	2.96	2.94	2.62	4.87	4.82	4.32	7.39	7.28	6.55	9.40	10.39	9.38
	1.29	1.28	1.07	2.41	2.40	2.05	3.98	3.94	3.39	6.04	5.95	5.14	8.63	8.48	7.35
27	1.53	1.52	1.34	2.86	2.84	2.52	4.68	4.64	4.16	7.08	6.99	6.29	10.15	9.96	8.94
	1.25	1.26	1.06	2.35	2.35	2.01	3.88	3.84	3.32	5.87	5.79	5.02	8.35	8.26	7.14
24	1.48	1.47	1.28	2.75	2.74	2.43	4.49	4.45	3.98	6.76	6.68	6.01	9.62	9.51	8.60
	1.25	1.24	1.05	2.31	2.30	1.97	3.78	3.75	3.24	5.69	5.62	4.89	8.07	7.94	6.95
21	1.43	1.43	1.24	2.64	2.63	2.32	4.28	4.26	3.80	6.42	6.36	5.72	9.13	8.97	8.12
	1.22	1.22	1.03	2.26	2.25	1.93	3.67	3.65	3.15	5.50	5.45	4.75	7.80	7.68	6.79
18	1.37	1.39	1.20	2.52	2.52	2.22	4.07	4.05	3.60	6.07	6.03	5.40	8.57	8.51	7.68
	1.20	1.19	1.00	2.21	2.20	1.88	3.56	3.54	3.06	5.31	5.27	4.60	7.52	7.45	6.56
15	1.31	1.32	1.14	2.40	2.40	2.10	3.85	3.83	3.39	5.70	5.67	5.06	7.99	7.98	7.16
	1.19	1.17	1.00	2.15	2.14	1.84	3.44	3.43	2.97	5.10	5.08	4.43	7.17	7.13	6.26
12	1.26	1.24	1.08	2.27	2.27	1.97	3.61	3.61	3.16	5.31	5.30	4.69	7.40	7.38	6.60
	1.16	1.14	0.98	2.09	2.09	1.79	3.36	3.32	2.87	4.89	4.88	4.26	6.60	6.82	6.00
9	1.19	1.17	1.02	2.12	2.12	1.83	3.36	3.35	2.92	4.91	4.90	4.30	6.77	6.81	5.98
	1.13	1.13	0.96	2.02	2.02	1.73	3.20	3.20	2.76	4.68	4.67	4.07	6.47	6.43	5.68
6	1.10	1.09	0.93	1.97	1.97	1.69	3.09	3.09	2.68	4.49	4.49	3.91	6.20	6.21	5.39
	1.09	1.09	0.93	1.95	1.95	1.67	3.06	3.06	2.65	4.45	4.44	3.86	6.15	6.11	5.29
3	0.70	0.70	0.60	1.25	1.25	1.08	1.96	1.96	1.70	2.83	2.83	2.47	3.86	3.86	3.40
	0.70	0.70	0.60	1.25	1.25	1.08	1.96	1.96	1.70	2.83	2.83	2.47	3.86	3.86	3.40



Notes to Roof Pressure Table 207B.6-2:

1. From table for Exposure C, V , h and roof slope, determine roof pressure p_h for each roof zone shown in the figures for the applicable roof form. For other exposures B or D, multiply pressures from table by appropriate exposure adjustment factor as determined from figure below.
2. Where two load cases are shown, both load cases shall be investigated. Load case 2 is required to investigate maximum overturning on the building from roof pressures shown.
3. Apply along-wind net wall pressures to the projected area of the building walls in the direction of the wind and apply exterior side wall pressures to the projected area of the building walls normal to the direction of the wind acting outward, simultaneously with the roof pressures from Table 207B.6-2.
4. Where a value of zero is shown in the tables for the flat roof case, it is provided for the purpose of interpolation.
5. Interpolation between V , h and roof slope is permitted.

Figure 207B.6-2
Application of Roof Pressures Wind Pressures - Roofs, $h \leq 48$ m
Enclosed Simple Diaphragm Buildings

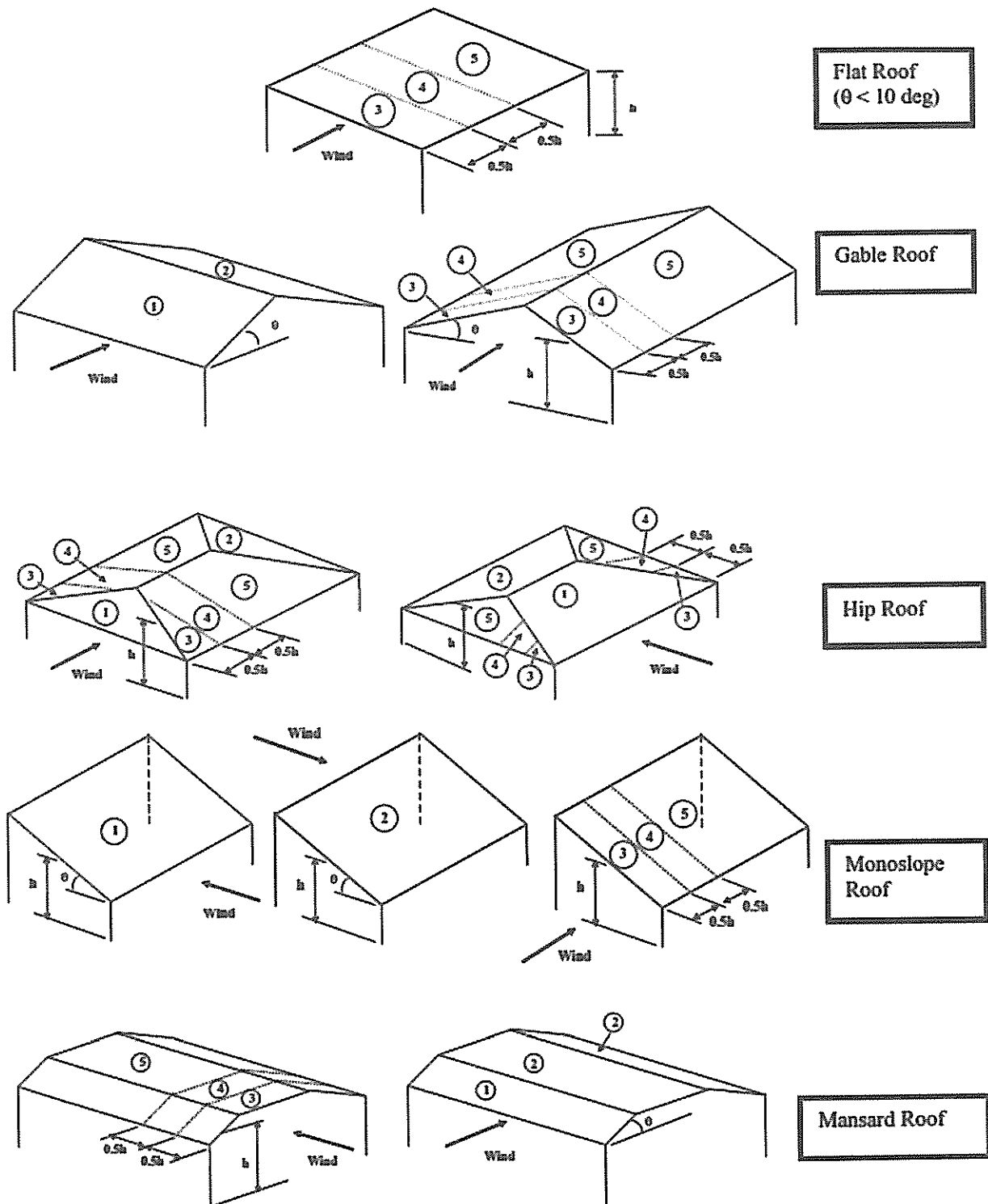


Figure 207B.6-2
Application of Roof Pressures Wind Pressures - Roofs, $h \leq 48$ m
Enclosed Simple Diaphragm Buildings

Table 207B.6-2
 MWFRS - Part 2: Wind Loads – Roof (kN/m²)
 MWFRS – Roof, $V = 150 - 250$ kph, $h = 3 - 12$ m
 Exposure C

h (m)	V (kph)	Roof Slope	Load Case	150					200					250				
				Zone					Zone					Zone				
				1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
12	Flat < 2:12 (9.46°)		1	NA	NA	-1.00	-0.89	-0.23	NA	NA	-1.78	-1.58	-1.30	NA	NA	-2.77	-2.47	-2.03
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)		1	-0.99	-0.48	-1.00	-0.89	-0.73	-1.76	-1.18	-1.78	-1.58	-1.30	-2.72	-1.91	-2.77	-2.47	-2.03
			2	0.14	-0.20	0.00	0.00	0.00	0.25	-0.35	0.00	0.00	0.00	0.39	-0.55	0.00	0.00	0.00
	4:12 (18.4°)		1	-0.80	-0.66	-1.00	-0.89	-0.73	-1.43	-1.16	-1.78	-1.58	-1.30	-2.24	-1.81	-2.77	-2.47	-2.03
			2	0.43	-0.09	0.00	0.00	0.00	0.50	-0.49	0.00	0.00	0.00	0.77	-0.79	0.00	0.00	0.00
	5:12 (22.6°)		1	-0.64	-0.66	-1.00	-0.89	-0.73	-1.15	-1.16	-1.78	-1.58	-1.30	-1.80	-1.81	-2.77	-2.47	-2.03
			2	0.37	-3.01	0.00	0.00	0.00	NA	-0.55	0.00	0.00	0.00	1.03	-0.86	0.00	0.00	0.00
	6:12 (26.6°)		1	-0.52	-0.66	-1.14	-0.89	-0.73	-0.92	-1.16	-1.78	-1.58	-1.30	-1.44	-1.81	-2.77	-2.47	-2.03
			2	0.41	-0.32	0.00	0.00	0.00	0.73	-0.55	0.00	0.00	0.00	1.14	-0.86	0.00	0.00	0.00
	9:12 (36.9°)		1	2.87	-0.66	-0.98	-0.89	-0.73	-0.07	-1.16	-1.78	-1.58	-1.30	-0.83	-1.81	-2.77	-2.47	-2.03
			2	0.49	-0.32	0.00	0.00	0.00	0.87	-0.55	0.00	0.00	0.00	0.95	-0.86	0.00	0.00	0.00
9	Flat < 2:12 (9.46°)		1	NA	NA	-0.93	-0.83	6.55	NA	NA	-1.67	-1.49	-1.22	NA	NA	-2.61	-2.32	-1.90
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)		1	-0.92	-0.45	-0.93	-0.83	-0.68	-1.64	-1.11	-1.67	-1.49	-1.22	-2.56	-1.80	-2.61	-2.32	-1.90
			2	0.14	-0.19	0.00	0.00	0.00	0.24	-0.33	0.00	0.00	0.00	0.37	-0.52	0.00	0.00	0.00
	4:12 (18.4°)		1	-0.75	-0.60	-0.93	-0.83	-0.68	-1.34	-1.08	-1.67	-1.49	-1.22	-2.10	-1.66	-2.61	-2.32	-1.90
			2	2.55	2.56	0.00	0.00	0.00	0.47	-0.31	0.00	0.00	0.00	0.73	-0.75	0.00	0.00	0.00
	5:12 (22.6°)		1	-0.60	-0.60	-0.93	-0.83	-0.68	-1.08	-1.08	-1.67	-1.49	-1.22	-1.69	-1.70	-2.61	-2.32	-1.90
			2	0.36	-0.29	0.00	0.00	0.00	0.62	-0.52	0.00	0.00	0.00	0.97	-0.81	0.00	0.00	0.00
	6:12 (26.6°)		1	-0.49	-0.60	-0.93	-0.83	-0.68	-0.87	-1.08	-1.67	-1.49	-1.22	-1.35	-1.70	-2.61	-2.32	-1.90
			2	0.39	-0.29	0.00	0.00	0.00	0.68	-0.52	0.00	0.00	0.00	1.07	-0.81	0.00	0.00	0.00
	9:12 (36.9°)		1	-0.29	-0.60	-0.93	-0.83	-0.68	-0.50	-1.08	-1.67	-1.49	-1.22	-0.78	-1.70	-2.61	-2.32	-1.90
			2	0.45	-0.29	0.00	0.00	0.00	0.82	-0.52	0.00	0.00	0.00	0.90	-0.81	0.00	0.00	0.00
6	Flat < 2:12 (9.46°)		1	NA	NA	-0.87	-0.76	-0.63	NA	NA	-1.54	-1.37	-1.12	NA	NA	-2.40	-2.14	-1.63
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)		1	-0.85	-0.43	-0.87	-0.76	-0.63	-1.50	-1.02	-1.54	-1.37	-1.12	-2.35	-1.65	-2.40	-2.14	-1.63
			2	0.13	-0.17	0.00	0.00	0.00	0.22	-0.31	0.00	0.00	0.00	0.34	-0.48	0.00	0.00	0.00
	4:12 (18.4°)		1	-0.69	-0.56	-0.87	-0.76	-0.63	-1.24	-0.94	-1.54	-1.37	-1.12	-1.93	-1.00	-2.40	-2.14	-1.63
			2	0.23	-0.24	0.00	0.00	0.00	0.43	-0.44	0.00	0.00	0.00	0.67	-0.68	0.00	0.00	0.00
	5:12 (22.6°)		1	-0.56	-0.56	-0.87	-0.76	-0.63	-0.99	-1.00	-1.54	-1.37	-1.12	-1.55	-1.56	-2.40	-2.14	-1.63
			2	0.33	-0.26	0.00	0.00	0.00	0.57	-0.48	0.00	0.00	0.00	0.89	-0.75	0.00	0.00	0.00
	6:12 (26.6°)		1	-0.45	-0.56	-0.87	-0.76	-0.63	-0.80	-1.00	-1.54	-1.37	-1.12	-1.25	-1.56	-2.40	-2.14	-1.63
			2	0.35	-0.26	0.00	0.00	0.00	0.63	-0.48	0.00	0.00	0.00	0.98	-0.75	0.00	0.00	0.00
	9:12 (36.9°)		1	-0.26	-0.56	-0.87	-0.76	-0.63	-0.46	-1.00	-1.54	-1.37	-1.12	-0.72	-1.56	-2.40	-2.14	-1.63
			2	0.41	-0.26	0.00	0.00	0.00	0.75	-0.48	0.00	0.00	0.00	0.83	-0.75	0.00	0.00	0.00
3	Flat < 2:12 (9.46°)		1	NA	NA	-0.76	-0.67	-0.56	NA	NA	-1.35	-1.21	-0.99	NA	NA	-2.12	-1.89	2.22
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)		1	-0.73	-0.36	-0.76	-0.67	-0.56	-1.33	-0.91	-1.35	-1.21	-0.99	-2.08	-1.46	-2.12	-1.89	2.22
			2	0.11	-0.14	0.00	0.00	0.00	0.19	-0.27	0.00	0.00	0.00	0.30	-0.42	0.00	0.00	0.00
	4:12 (18.4°)		1	-0.61	-0.47	-0.76	-0.67	-0.56	-1.09	0.88	-1.35	-1.21	-0.99	-1.70	-2.16	-2.12	-1.89	2.22
			2	0.22	-0.23	0.00	0.00	0.00	0.38	-0.39	0.00	0.00	0.00	0.59	-0.60	0.00	0.00	0.00
	5:12 (22.6°)		1	-0.47	-0.47	-0.76	-0.67	-0.56	-0.87	-0.88	-1.35	-1.21	-0.99	-1.37	-1.38	-2.12	-1.89	2.22
			2	0.28	-0.25	0.00	0.00	0.00	0.51	-0.42	0.00	0.00	0.00	0.79	-0.66	0.00	0.00	0.00
	6:12 (26.6°)		1	-0.38	-0.47	-0.76	-0.67	-0.56	-0.71	-0.88	-1.35	-1.21	-0.99	-1.10	-1.38	-2.12	-1.89	2.22
			2	0.32	-0.25	0.00	0.00	0.00	0.56	-0.42	0.00	0.00	0.00	0.87	-0.66	0.00	0.00	0.00
	9:12 (36.9°)		1	-0.21	-0.47	-0.76	-0.67	-0.56	-0.41	-0.88	-1.35	-1.21	-0.99	-0.63	-1.38	-2.12	-1.89	2.22
			2	0.38	-0.25	0.00	0.00	0.00	0.66	-0.42	0.00	0.00	0.00	0.73	-0.66	0.00	0.00	0.00
	12:12 (45.0°)		1	-0.12	-0.47	-0.76	-0.67	-0.56	-0.48	-0.88	-1.35	-1.21	-0.99	-0.36	-1.85	-2.12	-1.89	2.22
			2	0.38	-0.25	0.00	0.00	0.00	0.66	-0.42	0.00	0.00	0.00	1.04	-0.18	0.00	0.00	0.00

Table 207B.6-2
 MWFRS - Part 2: Wind Loads – Roof (kN/m²)
 MWFRS – Roof, $V = 300 - 350$ kph, $h = 3 - 12$ m
 Exposure C

V (kph)		Load Case	300					350				
h (m)	Roof Slope		Zone					Zone				
12	Flat < 2:12 (9.46°)	1	NA	NA	-3.99	-3.56	-2.92	NA	NA	-2.38	-2.12	-1.74
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-3.92	-2.66	-3.99	-3.56	-2.92	-2.33	-1.58	-2.38	-2.12	-1.74
		2	0.56	-0.79	0.00	0.00	0.00	0.34	-0.47	0.00	0.00	0.00
	4:12 (18.4°)	1	-3.22	-2.60	-3.99	-3.56	-2.92	-1.91	-1.55	-2.38	-2.12	-1.74
		2	1.12	-1.14	0.00	0.00	0.00	0.66	-0.68	0.00	0.00	0.00
	5:12 (22.6°)	1	-2.58	-2.60	-3.99	-3.56	-2.92	-1.54	-1.55	-2.38	-2.12	-1.74
		2	1.48	-1.24	0.00	0.00	0.00	0.88	-0.74	0.00	0.00	0.00
	6:12 (26.6°)	1	-2.07	-2.60	-3.99	-3.56	-2.92	-1.23	-1.55	-2.38	-2.12	-1.74
		2	1.64	-1.24	0.00	0.00	0.00	0.98	-0.74	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.20	-2.60	-3.99	-3.56	-2.92	-0.72	-1.55	-2.38	-2.12	-1.74
		2	1.96	-1.24	0.00	0.00	0.00	1.07	-0.68	0.00	0.00	0.00
9	Flat < 2:12 (9.46°)	1	NA	NA	-3.75	-3.34	-2.74	NA	NA	-2.34	-2.09	-1.71
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-3.68	-2.50	-3.75	-3.34	-2.74	-2.30	-1.56	-2.34	-2.09	-1.71
		2	0.53	-0.75	0.00	0.00	0.00	0.33	-0.47	0.00	0.00	0.00
	4:12 (18.4°)	1	-3.03	-2.44	-3.75	-3.34	-2.74	-1.89	-1.53	-2.34	-2.09	-1.71
		2	1.05	-1.07	0.00	0.00	0.00	0.65	-0.67	0.00	0.00	0.00
	5:12 (22.6°)	1	-2.43	-2.44	-3.75	-3.34	-2.74	-1.52	-1.53	-2.34	-2.09	-1.71
		2	1.40	-1.17	0.00	0.00	0.00	0.68	-0.73	0.00	0.00	0.00
	6:12 (26.6°)	1	-1.95	-2.44	-3.75	-3.34	-2.74	-1.22	-1.53	-2.34	-2.09	-1.71
		2	1.54	-1.17	0.00	0.00	0.00	0.96	-0.73	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.13	-2.44	-3.75	-3.34	-2.74	-0.71	-1.53	-2.34	-2.09	-1.71
		2	1.84	-1.17	0.00	0.00	0.00	1.15	-0.73	0.00	0.00	0.00
6	Flat < 2:12 (9.46°)	1	NA	NA	-3.45	-3.08	-2.52	NA	NA	-2.31	-2.06	-1.69
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-3.38	-2.30	-3.45	-3.08	-2.52	-2.27	-1.54	-2.31	-2.06	-1.69
		2	0.49	-0.69	0.00	0.00	0.00	0.33	-0.35	0.00	0.00	0.00
	4:12 (18.4°)	1	-2.78	-2.25	-3.45	-3.08	-2.52	-1.86	-1.50	-2.31	-2.06	-1.69
		2	0.96	-0.99	0.00	0.00	0.00	0.65	-0.66	0.00	0.00	0.00
	5:12 (22.6°)	1	-2.23	-2.25	-3.45	-3.08	-2.52	-1.49	-1.50	-2.31	-2.06	-1.69
		2	1.28	-1.07	0.00	0.00	0.00	0.24	-0.72	0.00	0.00	0.00
	6:12 (26.6°)	1	-1.79	-2.25	-3.45	-3.08	-2.52	-1.20	-1.50	-2.31	-2.06	-1.69
		2	1.42	-1.07	0.00	0.00	0.00	0.95	-0.72	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.04	-2.25	-3.45	-3.08	-2.52	-0.70	-1.50	-2.31	-2.06	-1.69
		2	1.69	-1.07	0.00	0.00	0.00	1.13	-0.72	0.00	0.00	0.00
3	Flat < 2:12 (9.46°)	1	NA	NA	-3.05	-2.72	-2.23	NA	NA	-2.27	-2.03	-1.66
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-2.99	-2.03	-3.05	-2.72	-2.23	-2.23	-1.52	-2.27	-2.03	-1.66
		2	0.43	-0.61	0.00	0.00	0.00	0.32	-0.07	0.00	0.00	0.00
	4:12 (18.4°)	1	-2.46	-1.98	-3.05	-2.72	-2.23	-1.83	-1.48	-2.27	-2.03	-1.66
		2	0.85	-0.87	0.00	0.00	0.00	0.64	-0.65	0.00	0.00	0.00
	5:12 (22.6°)	1	-1.98	-1.98	-3.05	-2.72	-2.23	-1.47	-1.48	-2.27	-2.03	-1.66
		2	1.13	-0.95	0.00	0.00	0.00	0.85	-0.71	0.00	0.00	0.00
	6:12 (26.6°)	1	-1.59	-1.98	-3.05	-2.72	-2.23	-1.18	-1.48	-2.27	-2.03	-1.66
		2	1.25	-0.95	0.00	0.00	0.00	0.93	-0.71	0.00	0.00	0.00
	9:12 (36.9°)	1	-0.92	-1.98	-3.05	-2.72	-2.23	-0.68	-1.48	-2.27	-2.03	-1.66
		2	1.49	-0.95	0.00	0.00	0.00	1.12	-0.71	0.00	0.00	0.00
	12:12 (45.0°)	1	-0.52	-0.44	-3.05	-2.72	-2.23	-0.39	-1.48	-2.27	-2.03	-1.66
		2	1.49	-0.95	0.00	0.00	0.00	19.8	-12.5	0.0	0.0	0.0

Table 207B.6-2
 MWFRS - Part 2: Wind Loads – Roof (kN/m²)
 MWFRS – Roof, $V = 150 - 250$ kph, $h = 15 - 24$ m
 Exposure C

h (m)	V (kph)	Roof Slope	Load Case	150					200					250				
				Zone					Zone					Zone				
				1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
24	Flat < 2:12 (9.46°)		1	NA	NA	-1.16	-1.03	-0.84	NA	NA	-2.05	-1.83	-1.50	NA	NA	-3.21	-2.86	-2.34
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)		1	-1.14	-0.57	-1.16	-1.03	-0.84	-2.02	-1.37	-2.05	-1.83	-1.50	-3.15	-2.21	-3.21	-2.86	-2.34
			2	0.17	-0.23	0.00	0.00	0.00	0.29	-0.41	0.00	0.00	0.00	0.45	-0.64	0.00	0.00	0.00
	4:12 (18.4°)		1	-8.45	-0.74	-1.16	-1.03	-0.84	-1.66	-1.34	-2.05	-1.83	-1.50	-2.59	-2.09	-3.21	-2.86	-2.34
			2	0.32	-0.33	0.00	0.00	0.00	0.57	-0.59	0.00	0.00	0.00	0.90	-0.92	0.00	0.00	0.00
	5:12 (22.6°)		1	-0.75	-0.74	-1.16	-1.03	-0.84	-1.33	-1.34	-2.05	-1.83	-1.50	-2.08	-2.09	-3.21	-2.86	-2.34
			2	0.42	-0.36	0.00	0.00	0.00	0.76	-0.64	0.00	0.00	0.00	1.19	-1.00	0.00	0.00	0.00
	6:12 (26.6°)		1	-0.60	-0.74	-1.16	-1.03	-0.84	-1.00	-1.34	-2.05	-1.83	-1.50	-1.67	-2.09	-3.21	-2.86	-2.34
			2	0.47	-0.36	0.00	0.00	0.00	0.84	-0.64	0.00	0.00	0.00	1.32	-1.00	0.00	0.00	0.00
	9:12 (36.9°)		1	-0.34	-0.74	-1.16	-1.03	-0.84	-0.62	-1.34	-2.05	-1.83	-1.50	-0.97	-2.09	-3.21	-2.86	-2.34
			2	NA	-0.36	0.00	0.00	0.00	1.01	-0.64	0.00	0.00	0.00	1.11	-1.00	0.00	0.00	0.00
21	Flat < 2:12 (9.46°)		1	-0.19	-0.74	-1.16	-1.03	-0.84	-0.35	-1.34	-2.05	-1.83	-1.50	-0.54	-2.09	-3.21	-2.86	-2.34
			2	0.56	-0.36	0.00	0.00	0.00	1.01	-0.64	0.00	0.00	0.00	1.57	-1.00	0.00	0.00	0.00
	3:12 (14.0°)		1	NA	NA	-1.13	-1.00	-0.83	NA	NA	-2.00	-1.78	-1.46	NA	NA	-3.12	-2.78	-2.28
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	4:12 (18.4°)		1	-1.10	-0.55	-1.13	-1.00	-0.83	-1.96	-1.33	-2.00	-1.78	-1.46	-3.06	-2.15	-3.12	-2.78	-2.28
			2	0.15	-0.22	0.00	0.00	0.00	0.28	-0.40	0.00	0.00	0.00	0.44	-0.62	0.00	0.00	0.00
	5:12 (22.6°)		1	-0.90	-0.73	-1.13	-1.00	-0.83	-1.61	-1.30	-2.00	-1.78	-1.46	-2.51	-2.03	-3.12	-2.78	-2.28
			2	0.32	-0.33	0.00	0.00	0.00	0.56	-0.57	0.00	0.00	0.00	0.87	-0.89	0.00	0.00	0.00
	6:12 (26.6°)		1	-0.72	-0.73	-1.13	-1.00	-0.83	-1.29	-1.30	-2.00	-1.78	-1.46	-2.02	-2.03	-3.12	-2.78	-2.28
			2	0.42	-0.36	0.00	0.00	0.00	0.74	-0.62	0.00	0.00	0.00	1.16	-0.97	0.00	0.00	0.00
	9:12 (36.9°)		1	-0.58	-0.73	-1.13	-1.00	-0.83	-0.59	-1.30	-2.00	-1.78	-1.46	-1.62	-2.03	-3.12	-2.78	-2.28
			2	0.45	-0.36	0.00	0.00	0.00	0.82	-0.62	0.00	0.00	0.00	1.28	-0.97	0.00	0.00	0.00
18	Flat < 2:12 (9.46°)		1	-0.34	-0.73	-1.13	-1.00	-0.83	-0.60	-1.30	-2.00	-1.78	-1.46	-0.94	-2.03	-3.12	-2.78	-2.28
			2	0.55	-0.36	0.00	0.00	0.00	0.98	-0.62	0.00	0.00	0.00	1.07	-0.97	0.00	0.00	0.00
	3:12 (14.0°)		1	-0.19	-0.73	-1.13	-1.00	-0.83	-0.34	-1.30	-2.00	-1.78	-1.46	-0.53	-2.03	-3.12	-2.78	-2.28
			2	0.55	-0.36	0.00	0.00	0.00	0.98	-0.62	0.00	0.00	0.00	1.53	-0.97	0.00	0.00	0.00
	4:12 (18.4°)		1	NA	NA	-1.08	-0.97	-0.80	NA	NA	-1.93	-1.72	-1.41	NA	NA	-3.02	-2.69	-2.21
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	5:12 (22.6°)		1	-1.07	-0.53	-1.08	-0.97	-0.80	-1.90	-1.29	-1.93	-1.72	-1.41	-2.96	-2.08	-3.02	-2.69	-2.21
			2	0.16	-0.22	0.00	0.00	0.00	0.27	-0.38	0.00	0.00	0.00	0.43	-0.60	0.00	0.00	0.00
	6:12 (26.6°)		1	-0.87	-0.70	-1.08	-0.97	-0.80	-1.56	-1.26	-1.93	-1.72	-1.41	-2.44	-1.97	-3.02	-2.69	-2.21
			2	0.31	-0.32	0.00	0.00	0.00	0.54	-0.55	0.00	0.00	0.00	0.84	-0.86	0.00	0.00	0.00
	9:12 (36.9°)		1	-0.70	-0.70	-1.08	-0.97	-0.80	-1.25	-1.26	-1.93	-1.72	-1.41	-1.95	-1.97	-3.02	-2.69	-2.21
			2	0.40	-0.35	0.00	0.00	0.00	0.72	-0.60	0.00	0.00	0.00	1.12	-0.94	0.00	0.00	0.00
15	Flat < 2:12 (9.46°)		1	-0.57	-0.70	-1.08	-0.97	-0.80	-1.00	-1.26	-1.93	-1.72	-1.41	-1.57	-1.97	-3.02	-2.69	-2.21
			2	0.45	-0.34	0.00	0.00	0.00	0.79	-0.60	0.00	0.00	0.00	1.24	-0.94	0.00	0.00	0.00
	3:12 (14.0°)		1	-0.32	-0.70	-1.08	-0.97	-0.80	-0.58	-1.26	-1.93	-1.72	-1.41	-0.90	-1.97	-3.02	-2.69	-2.21
			2	0.54	-0.34	0.00	0.00	0.00	0.95	-0.60	0.00	0.00	0.00	1.04	-0.94	0.00	0.00	0.00
	4:12 (18.4°)		1	0.01	-0.70	-1.08	-0.97	-0.80	-0.33	-1.26	-1.93	-1.72	-1.41	-0.51	-1.97	-3.02	-2.69	-2.21
			2	0.54	-0.34	0.00	0.00	0.00	0.95	-0.60	0.00	0.00	0.00	1.48	-0.94	0.00	0.00	0.00
	5:12 (22.6°)		1	NA	NA	-1.04	-0.94	-0.76	NA	NA	-1.86	-1.66	-1.36	NA	NA	-2.91	-2.59	-2.12
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	6:12 (26.6°)		1	-1.04	-0.51	-1.04	-0.94	-0.76	-1.82	-1.24	-1.86	-1.66	-1.36	-2.85	-2.00	-2.91	-2.59	-2.12
			2	0.15	-0.22	0.00	0.00	0.00	0.26	-0.37	0.00	0.00	0.00	0.41	-0.58	0.00	0.00	0.00
	9:12 (36.9°)		1	-0.84	-0.69	-1.04	-0.94	-0.76	-1.50	-1.21	-1.86	-1.66	-1.36	-2.34	-1.89	-2.91	-2.59	-2.12
			2	0.29	-0.30	0.00	0.00	0.00	0.52	-0.53	0.00	0.00	0.00	0.81	-0.83	0.00	0.00	0.00
12	Flat < 2:12 (9.46°)		1	-0.68	-0.69	-1.04	-0.94	-0.76	-1.20	-1.21	-1.86	-1.66	-1.36	-1.88	-1.89	-2.91	-2.59	-2.12
			2	0.38	-0.67	0.00	0.00	0.00	NA	-0.58	0.00	0.00	0.00	1.08	-0.91	0.00	0.00	0.00
	3:12 (14.0°)		1	-0.56	-0.69	-1.05	-0.94	-0.76	-0.97	-1.21	-1.86	-1.66	-1.36	-1.51	-1.89	-2.91	-2.59	-2.12
			2	0.42	-0.32	0.00	0.00	0.00	0.76	-0.58	0.00	0.00	0.00	1.19	-0.91	0.00	0.00	0.00
	4:12 (18.4°)		1	-0.05	-0.69	-1.04	-0.94	-0.76	-0.52	-1.21	-1.86	-1.66	-1.36	-0.87	-1.89	-2.91	-2.59	-2.12
			2	0.51	-0.32	0.00	0.00	0.00	0.91	-0.58	0.00	0.00	0.00	1.00	-0.91	0.00	0.00	0.00
	5:12 (22.6°)		1	1.67	-0.69	-1.04	-0.94	-0.76	-0.32	-1.21	-1.86	-1.66	-1.36	-0.49	-1.89	-2.91	-2.59	-2.12
			2	0.51	-0.32	0.00	0.00	0.00	0.91	-0.58	0.00	0.00	0.00	1.42	-0.91	0.00	0.00	0.00
	6:12 (26.6°)		1	NA	NA	-1.04	-0.94	-0.76	NA	NA	-1.86	-1.66	-1.36	NA	NA	-2.91	-2.59	-2.12
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	9:12 (36.9°)		1	-1.04	-0.51	-1.04	-0.94	-0.76	-1.82	-1.24	-1.86	-1.66	-1.36	-2.85	-2.00	-2.91	-2.59	-2.12
			2	0.15	-0.22	0.00	0.00	0.00	0.26	-0.37	0.00	0.00	0.00	0.41	-0.58	0.00	0.00	0.00

Table 207B.6-2
 MWFRS - Part 2: Wind Loads – Roof (kN/m²)
 MWFRS – Roof, $V = 300 - 350$ kph, $h = 15 - 24$ m
 Exposure C

V (kph)		Load Case	300					350				
h (m)	Roof Slope		Zone					Zone				
			1	2	3	4	5	1	2	3	4	5
24	Flat < 2:12 (9.46°)	1	NA	NA	-4.62	-4.12	-3.38	NA	NA	-2.38	-2.12	-1.74
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-4.53	-3.08	-4.62	-4.12	-3.38	-2.33	-1.58	-2.38	-2.12	-1.74
		2	0.65	-0.92	0.00	0.00	0.00	0.34	-0.47	0.00	0.00	0.00
	4:12 (18.4°)	1	-3.73	-3.01	-4.62	-4.12	-3.38	-1.91	-1.55	-2.38	-2.12	-1.74
		2	1.29	-1.32	0.00	0.00	0.00	0.66	-0.68	0.00	0.00	0.00
	5:12 (22.6°)	1	-2.99	-3.01	-4.62	-4.12	-3.38	-1.54	-1.55	-2.38	-2.12	-1.74
		2	1.72	-1.37	0.00	0.00	0.00	0.88	-0.74	0.00	0.00	0.00
	6:12 (26.6°)	1	-2.44	-3.01	-4.62	-4.12	-3.38	-1.23	-1.55	-2.38	-2.12	-1.74
		2	1.89	-1.44	0.00	0.00	0.00	0.98	-0.74	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.39	-3.01	-4.62	-4.12	-3.38	-0.72	-1.55	-2.38	-2.12	-1.74
		2	2.27	-1.44	0.00	0.00	0.00	1.07	-0.68	0.00	0.00	0.00
21	Flat < 2:12 (9.46°)	1	NA	NA	-4.49	-4.00	-3.29	NA	NA	-2.34	-2.09	-1.71
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-4.41	-2.99	-4.49	-4.00	-3.29	-2.30	-1.56	-2.34	-2.09	-1.71
		2	0.63	-0.90	0.00	0.00	0.00	0.33	-0.47	0.00	0.00	0.00
	4:12 (18.4°)	1	-3.62	-2.92	-4.49	-4.00	-3.29	-1.89	-1.53	-2.34	-2.09	-1.71
		2	1.25	-1.29	0.00	0.00	0.00	0.65	-0.67	0.00	0.00	0.00
	5:12 (22.6°)	1	-2.91	-2.92	-4.49	-4.00	-3.29	-1.52	-1.53	-2.34	-2.09	-1.71
		2	1.67	-1.40	0.00	0.00	0.00	0.68	-0.73	0.00	0.00	0.00
	6:12 (26.6°)	1	-2.63	-2.92	-4.49	-4.00	-3.29	-1.22	-1.53	-2.34	-2.09	-1.71
		2	1.84	-1.39	0.00	0.00	0.00	0.96	-0.73	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.35	-2.92	-4.49	-4.00	-3.29	-0.71	-1.53	-2.34	-2.09	-1.71
		2	2.20	-1.39	0.00	0.00	0.00	1.15	-0.73	0.00	0.00	0.00
18	Flat < 2:12 (9.46°)	1	NA	NA	-4.35	-3.88	-3.18	NA	NA	-2.31	-2.06	-1.69
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-4.26	-2.90	-4.35	-3.88	-3.18	-2.27	-1.54	-2.31	-2.06	-1.69
		2	0.61	-0.87	0.00	0.00	0.00	0.33	-0.35	0.00	0.00	0.00
	4:12 (18.4°)	1	-3.51	-2.83	-4.35	-3.88	-3.18	-1.86	-1.50	-2.31	-2.06	-1.69
		2	1.21	-1.24	0.00	0.00	0.00	0.65	-0.66	0.00	0.00	0.00
	5:12 (22.6°)	1	-2.82	-2.83	-4.35	-3.88	-3.18	-1.49	-1.50	-2.31	-2.06	-1.69
		2	1.62	-1.35	0.00	0.00	0.00	0.24	-0.72	0.00	0.00	0.00
	6:12 (26.6°)	1	-2.26	-2.83	-4.35	-3.88	-3.18	-1.20	-1.50	-2.31	-2.06	-1.69
		2	1.78	-1.35	0.00	0.00	0.00	0.95	-0.72	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.31	-2.83	-4.35	-3.88	-3.18	-0.70	-1.50	-2.31	-2.06	-1.69
		2	2.13	-1.35	0.00	0.00	0.00	1.13	-0.72	0.00	0.00	0.00
15	Flat < 2:12 (9.46°)	1	NA	NA	-4.18	-3.73	-3.06	NA	NA	-2.27	-2.03	-1.66
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-4.10	-2.79	-4.18	-3.73	-3.06	-2.23	-1.52	-2.27	-2.03	-1.66
		2	0.59	-0.83	0.00	0.00	0.00	0.32	-0.07	0.00	0.00	0.00
	4:12 (18.4°)	1	-3.37	-2.72	-4.18	-3.73	-3.06	-1.83	-1.48	-2.27	-2.03	-1.66
		2	1.17	-1.20	0.00	0.00	0.00	0.64	-0.65	0.00	0.00	0.00
	5:12 (22.6°)	1	-2.71	-2.72	-4.18	-3.73	-3.06	-1.47	-1.48	-2.27	-2.03	-1.66
		2	1.56	-1.30	0.00	0.00	0.00	0.85	-0.71	0.00	0.00	0.00
	6:12 (26.6°)	1	-2.18	-2.72	-4.18	-3.73	-3.06	-1.18	-1.48	-2.27	-2.03	-1.66
		2	1.72	-1.30	0.00	0.00	0.00	0.93	-0.71	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.26	-2.72	-4.18	-3.73	-3.06	-0.68	-1.48	-2.27	-2.03	-1.66
		2	2.05	-1.30	0.00	0.00	0.00	1.12	-0.71	0.00	0.00	0.00
	12:12 (45.0°)	1	-0.71	-2.72	-4.18	-3.73	-3.06	-0.39	-1.48	-2.27	-2.03	-1.66
		2	2.05	-1.30	0.00	0.00	0.00	19.8	-12.5	0.0	0.0	0.0

Table 207B.6-2
 MWFRS - Part 2: Wind Loads – Roof (kN/m²)
 MWFRS – Roof, $V = 150 - 250$ kph, $h = 27 - 36$ m
 Exposure C

h (m)	Roof Slope	V (kph)	Load Case	150					200					250				
				Zone					Zone					Zone				
				1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
36	Flat < 2:12 (9.46°)	1	1	NA	NA	-1.26	-1.11	-0.92	NA	NA	-1.34	-1.20	-0.97	NA	NA	-2.79	-2.48	-2.04
			2	NA	NA	0.00	0.00	0.00	NA	NA	NA	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	1	-1.23	-0.62	-1.26	-1.11	-0.92	-1.31	-0.65	-1.34	-1.20	-0.97	-2.73	-1.93	-2.79	-2.48	-2.04
			2	0.18	-0.25	0.00	0.00	0.00	0.19	-0.27	NA	0.00	0.00	0.39	-0.56	0.00	0.00	0.00
	4:12 (18.4°)	1	1	-1.01	-0.82	-1.26	-1.11	-0.92	-1.08	-0.87	-1.34	-1.20	-0.97	-2.25	-1.81	-2.79	-2.48	-2.04
			2	0.35	-0.36	0.00	0.00	0.00	0.37	-0.37	NA	0.00	0.00	0.78	-0.80	0.00	0.00	0.00
	5:12 (22.6°)	1	1	-0.82	-0.82	-1.26	-1.11	-0.92	-0.86	-0.87	-1.34	-1.20	-0.97	-1.80	-1.81	-2.79	-2.48	-2.04
			2	0.46	-0.38	0.00	0.00	0.00	0.50	-0.41	NA	0.00	0.00	1.04	-0.87	0.00	0.00	0.00
	6:12 (26.6°)	1	1	-0.65	-0.82	-1.26	-1.11	-0.92	-0.69	-0.87	-1.34	-1.20	-0.97	-1.45	-1.81	-2.79	-2.48	-2.04
			2	-4.19	-0.38	0.00	0.00	0.00	0.55	-0.41	NA	0.00	0.00	1.14	-0.87	0.00	0.00	0.00
	9:12 (36.9°)	1	1	-0.37	-0.82	-1.26	-1.11	-0.92	-0.40	-0.87	-1.34	-1.20	-0.97	-0.84	-1.81	-2.79	-2.48	-2.04
			2	0.62	-0.38	0.00	0.00	0.00	0.65	-0.41	NA	0.00	0.00	1.37	-0.87	0.00	0.00	0.00
	12:12 (45.0°)	1	1	-0.21	-0.82	-1.26	-1.11	-0.92	-0.23	-0.87	-1.34	-1.20	-0.97	-0.47	-1.81	-2.79	-2.48	-2.04
			2	0.62	-0.38	0.00	0.00	0.00	0.65	-0.41	NA	0.00	0.00	1.37	-0.87	0.00	0.00	0.00
33	Flat < 2:12 (9.46°)	1	1	NA	NA	-1.23	-1.10	-0.89	NA	NA	-1.33	-1.18	-0.96	NA	NA	-2.73	-2.44	-2.00
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	1	-1.22	-0.60	-1.23	-1.10	-0.89	-1.28	-0.65	-1.33	-1.18	-0.96	-2.68	-1.90	-2.73	-2.44	-2.00
			2	0.17	-0.25	0.00	0.00	0.00	0.17	-0.26	0.00	0.00	0.00	0.39	-0.55	0.00	0.00	0.00
	4:12 (18.4°)	1	1	-1.00	-0.82	-1.23	-1.10	-0.89	-1.06	-0.85	-1.33	-1.18	-0.96	-2.21	-1.78	-2.73	-2.44	-2.00
			2	-0.02	-0.35	0.00	0.00	0.00	0.37	-0.38	0.00	0.00	0.00	0.77	-0.78	0.00	0.00	0.00
	5:12 (22.6°)	1	1	-0.80	-0.82	-1.23	-1.10	-0.89	-0.85	-0.85	-1.33	-1.18	-0.96	-1.77	-1.78	-2.73	-2.44	-2.00
			2	0.46	-0.38	0.00	0.00	0.00	0.49	-0.41	0.00	0.00	0.00	1.02	-0.85	0.00	0.00	0.00
	6:12 (26.6°)	1	1	-0.63	-0.82	-1.23	-1.10	-0.89	-0.68	-0.85	-1.33	-1.18	-0.96	-1.42	-1.78	-2.73	-2.44	-2.00
			2	0.51	-0.38	0.00	0.00	0.00	0.54	-0.41	0.00	0.00	0.00	1.12	-0.85	0.00	0.00	0.00
	9:12 (36.9°)	1	1	-0.36	-0.82	-1.23	-1.10	-0.89	-0.39	-0.85	-1.33	-1.18	-0.96	-0.82	-1.78	-2.73	-2.44	-2.00
			2	0.61	-0.38	0.00	0.00	0.00	0.65	-0.41	0.00	0.00	0.00	1.34	-0.85	0.00	0.00	0.00
	12:12 (45.0°)	1	1	-0.20	-0.82	-1.23	-1.10	-0.89	-0.22	-0.85	-1.33	-1.18	-0.96	-0.46	-1.78	-2.73	-2.44	-2.00
			2	0.61	-0.38	0.00	0.00	0.00	0.65	-0.41	0.00	0.00	0.00	1.34	-0.85	0.00	0.00	0.00
30	Flat < 2:12 (9.46°)	1	1	NA	NA	-1.21	-1.08	-0.89	NA	NA	-1.30	-1.16	-0.95	NA	NA	-2.68	-2.39	-1.96
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	1	-1.19	-0.59	-1.21	-1.08	-0.89	-1.27	-0.63	-1.30	-1.16	-0.95	-2.63	-1.86	-2.68	-2.39	-1.96
			2	0.17	-0.24	0.00	0.00	0.00	0.18	-0.26	0.00	0.00	0.00	0.38	-0.53	0.00	0.00	0.00
	4:12 (18.4°)	1	1	-0.98	-0.79	-1.21	-1.08	-0.89	-1.05	-0.84	-1.30	-1.16	-0.95	-2.16	-1.74	-2.68	-2.39	-1.96
			2	-1.41	-0.35	0.00	0.00	0.00	0.36	-0.37	0.00	0.00	0.00	0.75	-0.77	0.00	0.00	0.00
	5:12 (22.6°)	1	1	-0.79	-0.79	-1.21	-1.08	-0.89	-0.84	-0.84	-1.30	-1.16	-0.95	-1.74	-1.74	-2.68	-2.39	-1.96
			2	0.44	-0.38	0.00	0.00	0.00	0.48	-0.40	0.00	0.00	0.00	1.00	-0.84	0.00	0.00	0.00
	6:12 (26.6°)	1	1	-0.63	-0.79	-1.21	-1.08	-0.89	-0.68	-0.84	-1.30	-1.16	-0.95	-1.39	-1.74	-2.68	-2.39	-1.96
			2	0.50	-0.38	0.00	0.00	0.00	0.54	-0.40	0.00	0.00	0.00	1.10	-0.84	0.00	0.00	0.00
	9:12 (36.9°)	1	1	-0.36	-0.79	-1.21	-1.08	-0.89	-0.38	-0.84	-1.30	-1.16	-0.95	-0.81	-1.74	-2.68	-2.39	-1.96
			2	0.59	-0.38	0.00	0.00	0.00	0.64	-0.40	0.00	0.00	0.00	1.32	-0.84	0.00	0.00	0.00
	12:12 (45.0°)	1	1	-0.20	-0.79	-1.21	-1.08	-0.89	-0.22	-0.84	-1.30	-1.16	-0.95	-0.46	-1.74	-2.68	-2.39	-1.96
			2	0.59	-0.38	0.00	0.00	0.00	0.64	-0.40	0.00	0.00	0.00	1.32	-0.84	0.00	0.00	0.00
27	Flat < 2:12 (9.46°)	1	1	NA	NA	-1.17	-1.05	-0.87	NA	NA	-1.28	-1.14	-0.94	NA	NA	-2.72	-2.42	-1.99
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	1	-1.17	-0.57	-1.17	-1.05	-0.87	-1.25	-0.63	-1.28	-1.14	-0.94	-2.66	-1.88	-2.72	-2.42	-1.99
			2	0.18	-0.24	0.00	0.00	0.00	0.18	-0.25	0.00	0.00	0.00	0.38	-0.54	0.00	0.00	0.00
	4:12 (18.4°)	1	1	-2.20	-0.77	-1.17	-1.05	-0.87	-1.03	-0.83	-1.28	-1.14	-0.94	-2.19	-1.77	-2.72	-2.42	-1.99
			2	0.33	-0.34	0.00	0.00	0.00	0.35	-0.37	0.00	0.00	0.00	0.76	-0.78	0.00	0.00	0.00
	5:12 (22.6°)	1	1	-0.78	-0.77	-1.17	-1.05	-0.87	-0.84	-0.83	-1.28	-1.14	-0.94	-1.76	-1.77	-2.72	-2.42	-1.99
			2	0.44	-0.37	0.00	0.00	0.00	0.47	-0.39	0.00	0.00	0.00	1.01	-0.85	0.00	0.00	0.00
	6:12 (26.6°)	1	1	-0.63	-0.77	-1.17	-1.05	-0.87	-0.66	-0.83	-1.28	-1.14	-0.94	-1.41	-1.77	-2.72	-2.42	-1.99
			2	0.49	-0.37	0.00	0.00	0.00	-0.68	-0.39	0.00	0.00	0.00	1.12	-0.85	0.00	0.00	0.00
	9:12 (36.9°)	1	1	-0.35	-0.77	-1.17	-1.05	-0.87	-0.37	-0.83	-1.28	-1.14	-0.94	-0.82	-1.77	-2.72	-2.42	-1.99
			2	NA	-0.37	0.00	0.00	0.00	0.63	-0.39	0.00	0.00	0.00	1.26	-0.85	0.00	0.00	0.00
	12:12 (45.0°)	1	1	-0.21	-0.77	-1.17	-1.05	-0.87	-0.21	-0.83	-1.28	-1.14	-0.94	-0.46	-1.77	-2.72	-2.42	-1.99
			2	0.57	-0.37	0.00	0.00	0.00	13.1	-8.2	0.0	0.0	0.0	1.33	-0.85	0.00	0.00	0.00

Table 207B.6-2
 MWFRS - Part 2: Wind Loads – Roof (kN/m²)
 MWFRS – Roof, $V = 300 - 350$ kph, $h = 27 - 36$ m
 Exposure C

V (kph)		Load Case	300					350				
h (m)	Roof Slope		Zone					Zone				
			1	2	3	4	5	1	2	3	4	5
36	Flat < 2:12 (9.46°)	1	NA	NA	-5.03	-4.49	-3.68	NA	NA	-2.38	-2.12	-1.74
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-4.94	-3.36	-5.03	-4.49	-3.68	-2.33	-1.58	-2.38	-2.12	-1.74
		2	0.71	-1.00	0.00	0.00	0.00	0.34	-0.47	0.00	0.00	0.00
	4: 12 (18.4°)	1	-4.06	-3.28	-5.03	-4.49	-3.68	-1.91	-1.55	-2.38	-2.12	-1.74
		2	1.40	-1.44	0.00	0.00	0.00	0.66	-0.68	0.00	0.00	0.00
	5:12 (22.6°)	1	-3.26	-3.28	-5.03	-4.49	-3.68	-1.54	-1.55	-2.38	-2.12	-1.74
		2	1.87	-1.57	0.00	0.00	0.00	0.88	-0.74	0.00	0.00	0.00
	6:12 (26.6°)	1	-2.62	-3.28	-5.03	-4.49	-3.68	-1.23	-1.55	-2.38	-2.12	-1.74
		2	2.06	-1.57	0.00	0.00	0.00	0.98	-0.74	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.51	-3.28	-5.03	-4.49	-3.68	-0.72	-1.55	-2.38	-2.12	-1.74
		2	2.47	-1.57	0.00	0.00	0.00	1.07	-0.68	0.00	0.00	0.00
33	Flat < 2:12 (9.46°)	1	NA	NA	-4.94	-4.40	-3.61	NA	NA	-2.34	-2.09	-1.71
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-4.85	-3.30	-4.94	-4.40	-3.61	-2.30	-1.56	-2.34	-2.09	-1.71
		2	0.70	-0.98	0.00	0.00	0.00	0.33	-0.47	0.00	0.00	0.00
	4: 12 (18.4°)	1	-3.98	-3.22	-4.94	-4.40	-3.61	-1.89	-1.53	-2.34	-2.09	-1.71
		2	1.38	-1.41	0.00	0.00	0.00	0.65	-0.67	0.00	0.00	0.00
	5:12 (22.6°)	1	-3.20	-3.21	-4.94	-4.40	-3.61	-1.52	-1.53	-2.34	-2.09	-1.71
		2	1.84	-1.54	0.00	0.00	0.00	0.68	-0.73	0.00	0.00	0.00
	6:12 (26.6°)	1	-2.57	-3.22	-4.94	-4.40	-3.61	-1.22	-1.53	-2.34	-2.09	-1.71
		2	2.03	-1.54	0.00	0.00	0.00	0.96	-0.73	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.49	-3.22	-4.94	-4.40	-3.61	-0.71	-1.53	-2.34	-2.09	-1.71
		2	2.42	-1.54	0.00	0.00	0.00	1.15	-0.73	0.00	0.00	0.00
30	Flat < 2:12 (9.46°)	1	NA	NA	-4.84	-4.32	-3.54	NA	NA	-2.31	-2.06	-1.69
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-4.75	-3.23	-4.84	-4.32	-3.54	-2.27	-1.54	-2.31	-2.06	-1.69
		2	0.69	-0.96	0.00	0.00	0.00	0.33	-0.35	0.00	0.00	0.00
	4: 12 (18.4°)	1	-3.91	-3.15	-4.84	-4.32	-3.54	-1.86	-1.50	-2.31	-2.06	-1.69
		2	1.35	-1.38	0.00	0.00	0.00	0.65	-0.66	0.00	0.00	0.00
	5:12 (22.6°)	1	-3.13	-3.14	-4.84	-4.32	-3.54	-1.49	-1.50	-2.31	-2.06	-1.69
		2	1.80	-1.51	0.00	0.00	0.00	0.24	-0.72	0.00	0.00	0.00
	6:12 (26.6°)	1	-2.52	-3.15	-4.84	-4.32	-3.54	-1.20	-1.50	-2.31	-2.06	-1.69
		2	1.99	-1.50	0.00	0.00	0.00	0.95	-0.72	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.46	-3.15	-4.84	-4.32	-3.54	-0.70	-1.50	-2.31	-2.06	-1.69
		2	2.38	-1.51	0.00	0.00	0.00	1.13	-0.72	0.00	0.00	0.00
27	Flat < 2:12 (9.46°)	1	NA	NA	-4.74	-4.22	-3.46	NA	NA	-2.27	-2.03	-1.66
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-4.65	-3.16	-4.74	-4.22	-3.46	-2.23	-1.52	-2.27	-2.03	-1.66
		2	0.67	-0.94	0.00	0.00	0.00	0.32	-0.07	0.00	0.00	0.00
	4: 12 (18.4°)	1	-3.82	-3.08	-4.74	-4.22	-3.46	-1.83	-1.48	-2.27	-2.03	-1.66
		2	1.32	-1.35	0.00	0.00	0.00	0.64	-0.65	0.00	0.00	0.00
	5:12 (22.6°)	1	-3.07	-3.08	-4.74	-4.22	-3.46	-1.47	-1.48	-2.27	-2.03	-1.66
		2	1.76	-1.46	0.00	0.00	0.00	0.85	-0.71	0.00	0.00	0.00
	6:12 (26.6°)	1	-2.46	-3.08	-4.74	-4.22	-3.46	-1.18	-1.48	-2.27	-2.03	-1.66
		2	1.94	-1.46	0.00	0.00	0.00	0.93	-0.71	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.42	-3.08	-4.74	-4.22	-3.46	-0.68	-1.48	-2.27	-2.03	-1.66
		2	2.32	-1.47	0.00	0.00	0.00	1.12	-0.71	0.00	0.00	0.00
	12:12 (45.0°)	1	-0.80	-3.08	-4.74	-4.22	-3.46	-0.39	-1.48	-2.27	-2.03	-1.66
		2	2.32	-1.47	0.00	0.00	0.00	19.8	-12.5	0.0	0.0	0.0

Table 207B.6-2
 MWFRS - Part 2: Wind Loads – Roof (kN/m²)
 MWFRS – Roof, $V = 150 - 250$ kph, $h = 39 - 48$ m
 Exposure C

V (kph)		Load Case	150					200					250				
h (m)	Roof Slope		Zone					Zone					Zone				
			1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
48	Flat < 2:12 (9.46°)	1	NA	NA	-1.34	-1.20	-0.97	NA	NA	-2.38	-2.12	-1.74	NA	NA	-3.71	-2.57	-2.71
		2	NA	NA	NA	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)	1	-1.31	-0.65	-1.34	-1.20	-0.97	-2.33	-1.58	-2.38	-2.12	-1.74	-3.64	-2.56	-3.71	-2.57	-2.71
		2	0.19	-0.27	NA	0.00	0.00	0.34	-0.47	0.00	0.00	0.00	0.53	-0.74	0.00	0.00	0.00
	4:12 (18.4°)	1	-1.08	-0.87	-1.34	-1.20	-0.97	-1.91	-1.55	-2.38	-2.12	-1.74	-2.99	-2.42	-3.71	-2.57	-2.71
		2	0.37	-0.37	NA	0.00	0.00	0.66	-0.68	0.00	0.00	0.00	1.04	-0.83	0.00	0.00	0.00
	5:12 (22.6°)	1	-0.86	-0.87	-1.34	-1.20	-0.97	-1.54	-1.55	-2.38	-2.12	-1.74	-2.40	-2.42	-3.71	-2.57	-2.71
		2	0.50	-0.41	NA	0.00	0.00	0.88	-0.74	0.00	0.00	0.00	1.38	-1.16	0.00	0.00	0.00
	6:12 (26.6°)	1	-0.69	-0.87	-1.34	-1.20	-0.97	-1.23	-1.55	-2.38	-2.12	-1.74	-1.93	-2.42	-3.71	-2.57	-2.71
		2	0.55	-0.41	NA	0.00	0.00	0.98	-0.74	0.00	0.00	0.00	1.53	-1.16	0.00	0.00	0.00
	9:12 (36.9°)	1	-0.40	-0.87	-1.34	-1.20	-0.97	-0.72	-1.55	-2.38	-2.12	-1.74	-1.12	-2.42	-3.71	-2.57	-2.71
		2	0.65	-0.41	NA	0.00	0.00	1.07	-0.68	0.00	0.00	0.00	1.23	-1.06	0.00	0.00	0.00
45	Flat < 2:12 (9.46°)	1	-0.23	-0.87	-1.34	-1.20	-0.97	-0.40	-1.55	-2.38	-2.12	-1.74	-0.63	-2.42	-3.71	-2.57	-2.71
		2	0.65	-0.41	NA	0.00	0.00	1.17	-0.74	0.00	0.00	0.00	1.82	-1.16	0.00	0.00	0.00
	3:12 (14.0°)	1	NA	NA	-1.33	-1.18	-0.96	NA	NA	-2.34	-2.09	-1.71	NA	NA	-3.66	-1.08	-2.68
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	4:12 (18.4°)	1	-1.28	-0.65	-1.33	-1.18	-0.96	-2.30	-1.56	-2.34	-2.09	-1.71	-3.59	-2.52	-3.66	-1.08	-2.68
		2	0.17	-0.26	0.00	0.00	0.00	0.33	-0.47	0.00	0.00	0.00	0.52	-0.73	0.00	0.00	0.00
	5:12 (22.6°)	1	-1.06	-0.85	-1.33	-1.18	-0.96	-1.89	-1.53	-2.34	-2.09	-1.71	-2.96	-2.38	-3.66	-1.08	-2.68
		2	0.37	-0.38	0.00	0.00	0.00	0.65	-0.67	0.00	0.00	0.00	1.02	-0.35	0.00	0.00	0.00
	6:12 (26.6°)	1	-0.85	-0.85	-1.33	-1.18	-0.96	-1.52	-1.53	-2.34	-2.09	-1.71	-2.37	-2.38	-3.66	-1.08	-2.68
		2	0.49	-0.41	0.00	0.00	0.00	0.68	-0.73	0.00	0.00	0.00	1.36	-1.14	0.00	0.00	0.00
	9:12 (36.9°)	1	-0.68	-0.85	-1.33	-1.18	-0.96	-1.22	-1.53	-2.34	-2.09	-1.71	-1.90	-2.38	-3.66	-1.08	-2.68
		2	0.54	-0.41	0.00	0.00	0.00	0.96	-0.73	0.00	0.00	0.00	1.50	-1.14	0.00	0.00	0.00
42	Flat < 2:12 (9.46°)	1	-0.39	-0.85	-1.33	-1.18	-0.96	-0.71	-1.53	-2.34	-2.09	-1.71	-1.10	-2.38	-3.66	-1.08	-2.68
		2	0.65	-0.41	0.00	0.00	0.00	1.15	-0.73	0.00	0.00	0.00	1.26	-1.14	0.00	0.00	0.00
	3:12 (14.0°)	1	-0.22	-0.85	-1.33	-1.18	-0.96	-0.40	-1.53	-2.34	-2.09	-1.71	-0.62	-2.38	-3.66	-1.08	-2.68
		2	0.65	-0.41	0.00	0.00	0.00	1.15	-0.73	0.00	0.00	0.00	1.80	-1.14	0.00	0.00	0.00
	4:12 (18.4°)	1	NA	NA	-1.30	-1.16	-0.95	NA	NA	-2.31	-2.06	-1.69	NA	NA	-3.61	-3.22	-2.64
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	5:12 (22.6°)	1	-1.27	-0.63	-1.30	-1.16	-0.95	-2.27	-1.54	-2.31	-2.06	-1.69	-3.54	-2.49	-3.61	-3.22	-2.64
		2	0.18	-0.26	0.00	0.00	0.00	0.33	-0.35	0.00	0.00	0.00	0.51	-0.72	0.00	0.00	0.00
	6:12 (26.6°)	1	-1.05	-0.84	-1.30	-1.16	-0.95	-1.86	-1.50	-2.31	-2.06	-1.69	-2.91	-2.35	-3.61	-3.22	-2.64
		2	0.36	-0.37	0.00	0.00	0.00	0.65	-0.66	0.00	0.00	0.00	1.01	-1.03	0.00	0.00	0.00
	9:12 (36.9°)	1	-0.84	-0.84	-1.30	-1.16	-0.95	-1.49	-1.50	-2.31	-2.06	-1.69	-2.34	-2.35	-3.61	-3.22	-2.64
		2	0.48	-0.40	0.00	0.00	0.00	0.24	-0.72	0.00	0.00	0.00	1.34	-1.12	0.00	0.00	0.00
39	Flat < 2:12 (9.46°)	1	-0.68	-0.84	-1.30	-1.16	-0.95	-1.20	-1.50	-2.31	-2.06	-1.69	-1.88	-2.35	-3.61	-3.22	-2.64
		2	0.54	-0.40	0.00	0.00	0.00	0.95	-0.72	0.00	0.00	0.00	1.48	-1.12	0.00	0.00	0.00
	3:12 (14.0°)	1	-0.38	-0.84	-1.30	-1.16	-0.95	-0.70	-1.50	-2.31	-2.06	-1.69	-1.09	-2.35	-3.61	-3.22	-2.64
		2	0.64	-0.40	0.00	0.00	0.00	1.13	-0.72	0.00	0.00	0.00	1.24	-1.12	0.00	0.00	0.00
	4:12 (18.4°)	1	-0.22	-0.84	-1.30	-1.16	-0.95	-0.39	-1.50	-2.31	-2.06	-1.69	-0.61	-2.35	-3.61	-3.22	-2.64
		2	0.64	-0.40	0.00	0.00	0.00	1.09	-0.69	0.00	0.00	0.00	1.77	-1.12	0.00	0.00	0.00
	5:12 (22.6°)	1	NA	NA	-1.28	-1.14	-0.94	NA	NA	-2.27	-2.03	-1.66	NA	NA	-3.41	-3.04	-2.49
		2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	6:12 (26.6°)	1	-1.25	-0.63	-1.28	-1.14	-0.94	-2.23	-1.52	-2.27	-2.03	-1.66	-3.34	-2.35	-3.41	-3.04	-2.49
		2	0.18	-0.25	0.00	0.00	0.00	0.32	-0.07	0.00	0.00	0.00	0.48	-0.68	0.00	0.00	0.00
	9:12 (36.9°)	1	-1.03	-0.83	-1.28	-1.14	-0.94	-1.83	-1.48	-2.27	-2.03	-1.66	-2.75	-2.22	-3.41	-3.04	-2.49
		2	0.35	-0.37	0.00	0.00	0.00	0.64	-0.65	0.00	0.00	0.00	0.95	-0.97	0.00	0.00	0.00
39	Flat < 2:12 (9.46°)	1	-0.84	-0.83	-1.28	-1.14	-0.94	-1.47	-1.48	-2.27	-2.03	-1.66	-2.20	-2.22	-3.41	-3.04	-2.49
		2	0.47	-0.39	0.00	0.00	0.00	0.85	-0.71	0.00	0.00	0.00	1.27	-1.06	0.00	0.00	0.00
	3:12 (14.0°)	1	-0.66	-0.83	-1.28	-1.14	-0.94	-1.18	-1.48	-2.27	-2.03	-1.66	-1.77	-2.22	-3.41	-3.04	-2.49
		2	-0.68	-0.39	0.00	0.00	0.00	0.93	-0.71	0.00	0.00	0.00	1.40	-1.06	0.00	0.00	0.00
	4:12 (18.4°)	1	-0.37	-0.83	-1.28	-1.14	-0.94	-0.68	-1.48	-2.27	-2.03	-1.66	-1.03	-2.22	-3.41	-3.04	-2.49
		2	0.63	-0.39	0.00	0.00	0.00	1.12	-0.71	0.00	0.00	0.00	1.26	-1.06	0.00	0.00	0.00
	5:12 (22.6°)	1	-0.21	-0.83	-1.28	-1.14	-0.94	-0.39	-1.48	-2.27	-2.03	-1.66	-0.58	-2.22	-3.41	-3.04	-2.49
		2	13.1	-8.2	0.0	0.0	0.0	19.8	-12.5	0.0	0.0	0.0	1.67	-1.06	0.00	0.00	0.00

Table 207B.6-2
 MWFRS - Part 2: Wind Loads – Roof (kN/m²)
 MWFRS – Roof, $V = 300 - 350$ kph, $h = 39 - 48$ m
 Exposure C

h (m)	V (kph)	Roof Slope	Load Case	300					350				
				Zone					Zone				
				1	2	3	4	5	1	2	3	4	5
48	Flat < 2:12 (9.46°)		1	NA	NA	-5.35	-4.77	-3.91	NA	NA	-2.38	-2.12	-1.74
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)		1	-5.25	-3.56	-5.35	-4.77	-3.91	-2.33	-1.58	-2.38	-2.12	-1.74
			2	0.76	-1.06	0.00	0.00	0.00	0.34	-0.47	0.00	0.00	0.00
	4:12 (18.4°)		1	-4.31	-3.48	-5.35	-4.77	-3.91	-1.91	-1.55	-2.38	-2.12	-1.74
			2	1.49	-1.53	0.00	0.00	0.00	0.66	-0.68	0.00	0.00	0.00
	5:12 (22.6°)		1	-3.46	-3.43	-5.35	-4.77	-3.91	-1.54	-1.55	-2.38	-2.12	-1.74
			2	1.99	-1.66	0.00	0.00	0.00	0.88	-0.74	0.00	0.00	0.00
	6:12 (26.6°)		1	-2.78	-3.48	-5.35	-4.77	-3.91	-1.23	-1.55	-2.38	-2.12	-1.74
			2	2.20	-1.66	0.00	0.00	0.00	0.98	-0.74	0.00	0.00	0.00
	9:12 (36.9°)		1	-1.61	-3.48	-5.35	-4.77	-3.91	-0.72	-1.55	-2.38	-2.12	-1.74
			2	2.62	-1.66	0.00	0.00	0.00	1.07	-0.68	0.00	0.00	0.00
	12:12 (45.0°)		1	-0.91	-3.48	-5.35	-4.77	-3.91	-0.40	-1.55	-2.38	-2.12	-1.74
			2	2.62	-1.66	0.00	0.00	0.00	1.17	-0.74	0.00	0.00	0.00
45	Flat < 2:12 (9.46°)		1	NA	NA	-5.27	-4.70	-3.86	NA	NA	-2.34	-2.09	-1.71
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)		1	-5.18	-3.52	-5.27	-4.70	-3.86	-2.30	-1.56	-2.34	-2.09	-1.71
			2	0.75	-1.05	0.00	0.00	0.00	0.33	-0.47	0.00	0.00	0.00
	4:12 (18.4°)		1	-4.25	-3.43	-5.27	-4.70	-3.86	-1.89	-1.53	-2.34	-2.09	-1.71
			2	1.47	-1.51	0.00	0.00	0.00	0.65	-0.67	0.00	0.00	0.00
	5:12 (22.6°)		1	-3.41	-3.43	-5.27	-4.70	-3.86	-1.52	-1.53	-2.34	-2.09	-1.71
			2	1.96	-1.64	0.00	0.00	0.00	0.68	-0.73	0.00	0.00	0.00
	6:12 (26.6°)		1	-2.74	-3.43	-5.27	-4.70	-3.86	-1.22	-1.53	-2.34	-2.09	-1.71
			2	2.17	-1.64	0.00	0.00	0.00	0.96	-0.73	0.00	0.00	0.00
	9:12 (36.9°)		1	-1.59	-3.43	-5.27	-4.70	-3.86	-0.71	-1.53	-2.34	-2.09	-1.71
			2	2.59	-1.64	0.00	0.00	0.00	1.15	-0.73	0.00	0.00	0.00
	12:12 (45.0°)		1	-0.90	-3.43	-5.27	-4.70	-3.86	-0.40	-1.53	-2.34	-2.09	-1.71
			2	2.59	-1.64	0.00	0.00	0.00	1.15	-0.73	0.00	0.00	0.00
42	Flat < 2:12 (9.46°)		1	NA	NA	-5.20	-4.63	-3.80	NA	NA	-2.31	-2.06	-1.69
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)		1	-5.10	-3.47	-5.20	-4.63	-3.80	-2.27	-1.54	-2.31	-2.06	-1.69
			2	0.74	-1.03	0.00	0.00	0.00	0.33	-0.35	0.00	0.00	0.00
	4:12 (18.4°)		1	-4.19	-3.38	-5.20	-4.63	-3.80	-1.86	-1.50	-2.31	-2.06	-1.69
			2	1.45	-1.49	0.00	0.00	0.00	0.65	-0.66	0.00	0.00	0.00
	5:12 (22.6°)		1	-3.36	-3.38	-5.20	-4.63	-3.80	-1.49	-1.50	-2.31	-2.06	-1.69
			2	1.93	-1.62	0.00	0.00	0.00	0.24	-0.72	0.00	0.00	0.00
	6:12 (26.6°)		1	-2.70	-3.38	-5.20	-4.63	-3.80	-1.20	-1.50	-2.31	-2.06	-1.69
			2	2.13	-1.62	0.00	0.00	0.00	0.95	-0.72	0.00	0.00	0.00
	9:12 (36.9°)		1	-1.57	-3.38	-5.20	-4.63	-3.80	-0.70	-1.50	-2.31	-2.06	-1.69
			2	2.55	-1.62	0.00	0.00	0.00	1.13	-0.72	0.00	0.00	0.00
	12:12 (45.0°)		1	-0.88	-3.38	-5.20	-4.63	-3.80	-0.39	-1.50	-2.31	-2.06	-1.69
			2	2.55	-1.62	0.00	0.00	0.00	1.09	-0.69	0.00	0.00	0.00
39	Flat < 2:12 (9.46°)		1	NA	NA	-5.12	-4.56	-3.74	NA	NA	-2.27	-2.03	-1.66
			2	NA	NA	0.00	0.00	0.00	NA	NA	0.00	0.00	0.00
	3:12 (14.0°)		1	-5.02	-3.41	-5.12	-4.56	-3.74	-2.23	-1.52	-2.27	-2.03	-1.66
			2	0.72	-1.02	0.00	0.00	0.00	0.32	-0.07	0.00	0.00	0.00
	4:12 (18.4°)		1	-4.13	-3.33	-5.12	-4.56	-3.74	-1.83	-1.48	-2.27	-2.03	-1.66
			2	1.43	-1.46	0.00	0.00	0.00	0.64	-0.65	0.00	0.00	0.00
	5:12 (22.6°)		1	-3.31	-3.33	-5.12	-4.56	-3.74	-1.47	-1.48	-2.27	-2.03	-1.66
			2	1.90	-1.59	0.00	0.00	0.00	0.85	-0.71	0.00	0.00	0.00
	6:12 (26.6°)		1	-2.66	-3.33	-5.12	-4.56	-3.74	-1.18	-1.48	-2.27	-2.03	-1.66
			2	2.10	-1.59	0.00	0.00	0.00	0.93	-0.71	0.00	0.00	0.00
	9:12 (36.9°)		1	-1.54	-3.33	-5.12	-4.56	-3.74	-0.68	-1.48	-2.27	-2.03	-1.66
			2	2.51	-1.59	0.00	0.00	0.00	1.12	-0.71	0.00	0.00	0.00
	12:12 (45.0°)		1	-0.87	-3.33	-5.12	-4.56	-3.74	-0.39	-1.48	-2.27	-2.03	-1.66
			2	2.51	-1.59	0.00	0.00	0.00	1.12	-0.71	0.00	0.00	0.00

207C.3 Velocity Pressure

207C.3.1 Velocity Pressure Exposure Coefficient

Based on the Exposure Category determined in Section 207A.7.3, a velocity pressure exposure coefficient K_z or K_h , as applicable, shall be determined from Table 207C.3-1.

For a site located in a transition zone between exposure categories that is near to a change in ground surface roughness, intermediate values of K_z or K_h , between those shown in Table 207C.3-1, are permitted, provided that they are determined by a rational analysis method defined in the recognized literature.

Table 207C.2-1
Steps to Determine Wind Loads on MWFRS
Low-Rise Buildings

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|---------|--|
| Step 1: | Determine occupancy category of building or other structure, see Table 103-1 |
| Step 2: | Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C |
| Step 3: | Determine wind load parameters: <ul style="list-style-type: none"> ➤ Wind directionality factor, K_d, see Section 207A.6 and Table 207A.6-1 ➤ Exposure category B, C or D, see Section 207A.7 ➤ Topographic factor, K_{zt}, see Section 207A.8 and Figure 207A.8-1 ➤ Enclosure classification, see Section 207A.10 ➤ Internal pressure coefficient, (GC_{pi}), see Section 207A.11 and Table 207A.11-1 |
| Step 4: | Determine velocity pressure exposure coefficient, K_z or K_h , see Tables 207C.3-1 |
| Step 5: | Determine velocity pressure, q_z or q_h , see Equation 207C.3-1 |
| Step 6: | Determine external pressure coefficient, (GC_p) , using Figure 207C.4-1 for flat and gable roofs. |

User Note:

See Commentary Figure C207C.4-1 for guidance on hip roofs.

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| Step 7: | Calculate wind pressure, p , from Equation 207C.4-1 |
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Commentary:

See commentary to Section C207B.3.1.

207C.3.2 Velocity Pressure

Velocity pressure, q_z , evaluated at height z shall be calculated by the following equation:

$$q_z = 0.613K_zK_{zt}K_dV^2 \text{ (N/m}^2\text{); } V \text{ in m/s} \quad (207C.3-1)$$

where

- K_d = wind directionality factor, see Section 207A.6
- K_z = velocity pressure exposure coefficient, see Section 207C.3.1
- K_{zt} = topographic factor defined, see Section 207A.8.2
- V = basic wind speed, see Section 207A.5.1
- q_z = velocity pressure calculated using Equation 207C.3-1 at mean roof height h

The numerical coefficient 0.613 shall be used except where sufficient climatic data are available to justify the selection of a different value of this factor for a design application.

Commentary:

See commentary to Section C207B.3.2.

Loads on Main Wind-Force Resisting Systems:

The pressure coefficients for MWFRS are basically separated into two categories:

1. Directional Procedure for buildings of all heights (Figure 207B.4-1) as specified in Section 207B for buildings meeting the requirements specified therein.
2. Envelope Procedure for low-rise buildings (Figure 207C.4-1) as specified in Section 207C for buildings meeting the requirements specified therein.

In generating these coefficients, two distinctly different approaches were used. For the pressure coefficients given in Figure 207B.4-1, the more traditional approach was followed and the pressure coefficients reflect the actual loading on each surface of the building as a function of wind direction, namely, winds perpendicular or parallel to the ridge line.

For low-rise buildings, however, the values of (GC_{pf}) represent “pseudo” loading conditions that, when applied to the building, envelop the desired structural actions

(bending moment, shear, thrust) independent of wind direction. To capture all appropriate structural actions, the building must be designed for all wind directions by considering in turn each corner of the building as the windward or reference corner shown in the eight sketches of Figure 207C.4-1. At each corner, two load patterns are applied, one for each wind direction range. The end zone creates the required structural actions in the end frame or bracing. Note also that for all roof slopes, all eight load cases must be considered individually to determine the critical loading for a given structural assemblage or component thereof. Special attention should be given to roof members, such as trusses, which meet the definition of MWFRS but are not part of the lateral resisting system. When such members span at least from the eave to the ridge or support members spanning at least from eave to ridge, they are not required to be designed for the higher end zone loads under MWFRS. The interior zone loads should be applied. This is due to the enveloped nature of the loads for roof members.

To develop the appropriate “pseudo” values of (GC_{pf}), investigators at the University of Western Ontario (Davenport et al. 1978) used an approach that consisted essentially of permitting the building model to rotate in the wind tunnel through a full 360° while simultaneously monitoring the loading conditions on each of the surfaces (Figure C207C.4-1). Both Exposures B and C were considered. Using influence coefficients for rigid frames, it was possible to spatially average and time average the surface pressures to ascertain the maximum induced external force components to be resisted. More specifically, the following structural actions were evaluated:

1. Total uplift.
2. Total horizontal shear.
3. Bending moment at knees (two-hinged frame).
4. Bending moment at knees (three-hinged frame).
5. Bending moment at ridge (two-hinged frame).

The next step involved developing sets of “pseudo” pressure coefficients to generate loading conditions that would envelop the maximum induced force components to be resisted for all possible wind directions and exposures. Note, for example, that the wind azimuth producing the maximum bending moment at the knee would not necessarily produce the maximum total uplift. The maximum induced external force components determined for each of the preceding five categories were used to develop the coefficients. The end result was a set of coefficients that represent fictitious loading conditions but that conservatively envelop the maximum induced force

components (bending moment, shear, and thrust) to be resisted, independent of wind direction.

The original set of coefficients was generated for the framing of conventional pre-engineered buildings, that is, single-storey moment-resisting frames in one of the principal directions and bracing in the other principal direction. The approach was later extended to single-storey moment-resisting frames with interior columns (Kavanagh et al. 1983).

Subsequent wind tunnel studies (Isyumov and Case 1995) have shown that the (GC_{pf}) values of Figure 207C.4-1 are also applicable to low-rise buildings with structural systems other than moment-resisting frames. That work examined the instantaneous wind pressures on a low-rise building with a 4:12 pitched gable roof and the resulting wind-induced forces on its MWFRS. Two different MWFRS were evaluated. One consisted of shear walls and roof trusses at different spacing. The other had moment-resisting frames in one direction, positioned at the same spacing as the roof trusses, and diagonal wind bracing in the other direction. Wind tunnel tests were conducted for both Exposures B and C. The findings of this study showed that the (GC_{pf}) values of Figure 207C.4-1 provided satisfactory estimates of the wind forces for both types of structural systems. This work confirms the validity of Figure 207C.4-1, which reflects the combined action of wind pressures on different external surfaces of a building and thus takes advantage of spatial averaging.

In the original wind tunnel experiments, both B and C exposure terrains were checked. In these early experiments, Exposure B did not include nearby buildings. In general, the force components, bending moments, and so forth were found comparable in both exposures, although (GC_{pf}) values associated with Exposure B terrain would be higher than that for Exposure C terrain because of reduced velocity pressure in Exposure B terrain. The (GC_{pf}) values given in Figures 207C.4-1, 207E.4-1, 207E.4-2A, 207E.4-2B, 207E.4-2C, 207E.4-3, 207E.4-4, 207E.4-5A, 207E.4-5B, and 207E.4-6 are derived from wind tunnel studies modeled with Exposure C terrain. However, they may also be used in other exposures when the velocity pressure representing the appropriate exposure is used.

In comprehensive wind tunnel studies conducted by Ho at the University of Western Ontario (1992), it was determined that when low buildings ($h < 18$ m) are embedded in suburban terrain (Exposure B, which included nearby buildings), the pressures in most cases are lower than those currently used in existing standards and codes, although the values show a very large scatter because of high turbulence and many variables. The results

seem to indicate that some reduction in pressures for buildings located in Exposure B is justified. The ASCE Task Committee on Wind Loads believes it is desirable to design buildings for the exposure conditions consistent with the exposure designations defined in the standard. In the case of low buildings, the effect of the increased intensity of turbulence in rougher terrain (i.e., Exposure A or B vs. C) increases the local pressure coefficients. Beginning in NSCP 2001 (ASCE 7-98) the effect of the increased turbulence intensity on the loads is treated with the truncated profile. Using this approach, the actual building exposure is used and the profile truncation corrects for the underestimate in the loads that would be obtained otherwise.

Figure 207C.4-1 is most appropriate for low buildings with width greater than twice their height and a mean roof height that does not exceed 10 m. The original database included low buildings with width no greater than five times their eave height, and eave height did not exceed 10m. In the absence of more appropriate data, Figure 207C.4-1 may also be used for buildings with mean roof height that does not exceed the least horizontal dimension and is less than or equal to 18m. Beyond these extended limits, Figure 207B.4-1 should be used.

All the research used to develop and refine the low-rise building method for MWFRS loads was done on gable-roofed buildings. In the absence of research on hip-roofed buildings, the ASCE committee has developed a rational method of applying Figure 207C.4-1 to hip roofs based on its collective experience, intuition, and judgment. This suggested method is presented in Figure C207C.4-2.

Research (Isyumov 1982 and Isyumov and Case 2000) indicated that the low-rise method alone underestimates the amount of torsion caused by wind loads. In ASCE 7-02, Note 5 was added to Figure 207C.4-1 to account for this torsional effect and has been carried forward through subsequent editions. The reduction in loading on only 50 percent of the building results in a torsional load case without an increase in the predicted base shear for the building. The provision will have little or no effect on the design of MWFRS that have well-distributed resistance. However, it will impact the design of systems with centralized resistance, such as a single core in the center of the building. An illustration of the intent of the note on two of the eight load patterns is shown in Figure 207C.4-1. All eight patterns should be modified in this way as a separate set of load conditions in addition to the eight basic patterns.

Internal pressure coefficients (GC_{pi}) to be used for loads on MWFRS are given in Table 207A.11-1. The internal pressure load can be critical in one-storey moment-

resisting frames and in the top storey of a building where the MWFRS consists of moment resisting frames. Loading cases with positive and negative internal pressures should be considered. The internal pressure load cancels out in the determination of total lateral load and base shear. The designer can use judgment in the use of internal pressure loading for the MWFRS of high-rise buildings.

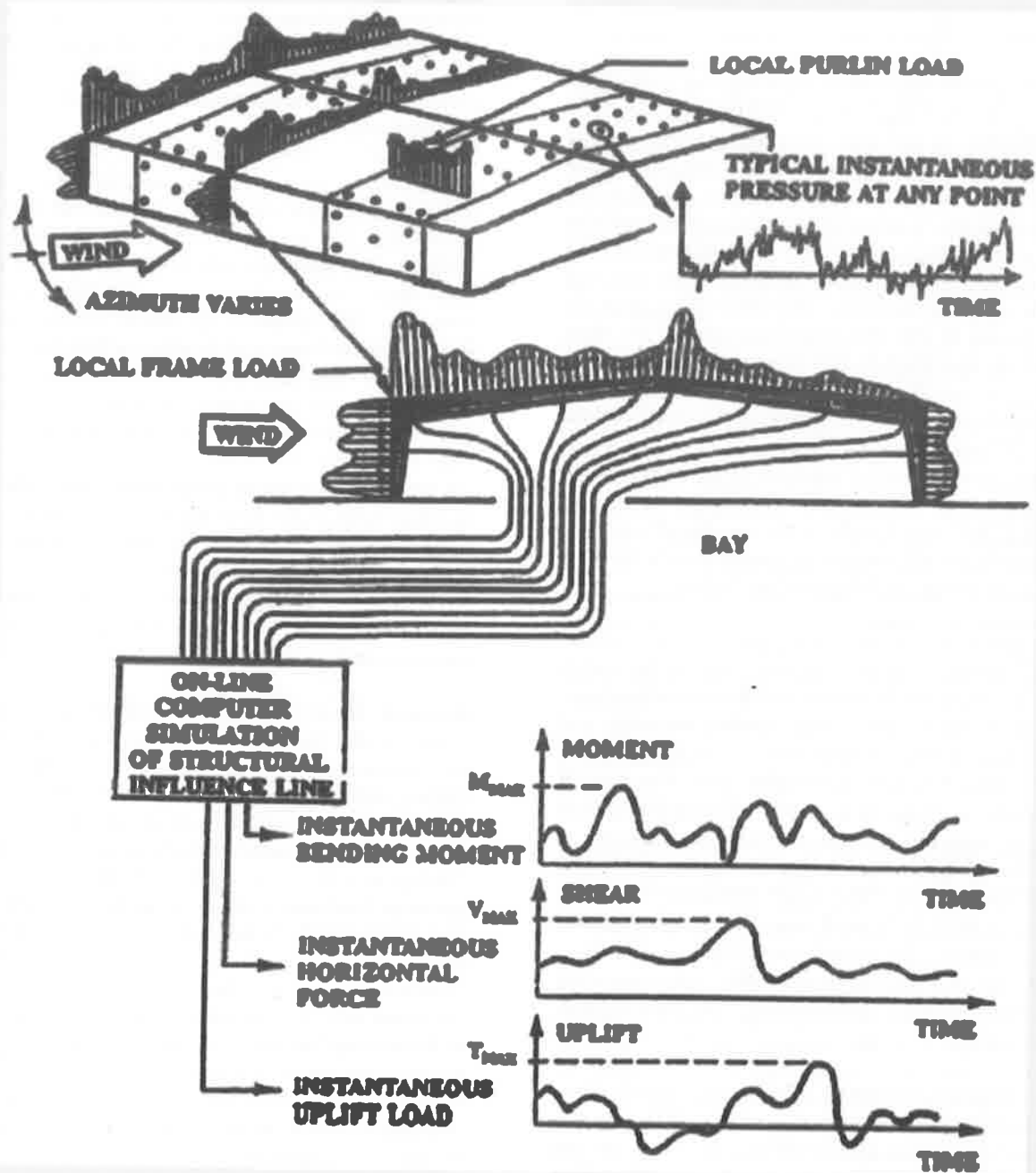


Figure C207C.4-1
Unsteady Wind Loads on Low Buildings for Given Wind Direction (After Ellingwood 1982)

207C.4 Wind Loads — Main Wind-Force Resisting System

207C.4.1 Design Wind Pressure for Low-Rise Buildings

Design wind pressures for the MWFRS of low-rise buildings shall be determined by the following equation:

$$p = q_h [(GC_{pf}) - (GC_{pi})] \text{ (N/m}^2\text{)} \quad (207C.4-1)$$

where

- q_h = velocity pressure evaluated at mean roof height h as defined in Section 207A.3
- (GC_{pf}) = external pressure coefficient from Figure 207C.4-1
- (GC_{pi}) = internal pressure coefficient from Table 207A.11-1

207C.4.1.1 External Pressure Coefficients (GC_{pf})

The combined gust effect factor and external pressure coefficients for low-rise buildings, (GC_{pf}), are not permitted to be separated.

207C.4.2 Parapets

The design wind pressure for the effect of parapets on MWFRS of low-rise buildings with flat, gable, or hip roofs shall be determined by the following equation:

$$p_p = q_p (GC_{pn}) \text{ (N/m}^2\text{)} \quad (207C.4-2)$$

where

- p_p = combined net pressure on the parapet due to the combination of the net pressures from the front and back parapet surfaces. Plus (and minus) signs signify net pressure acting toward (and away from) the front (exterior) side of the parapet
- q_p = velocity pressure evaluated at the top of the parapet
- (GC_{pn}) = combined net pressure coefficient
 - = +1.5 for windward parapet
 - = -1.0 for leeward parapet

Commentary:

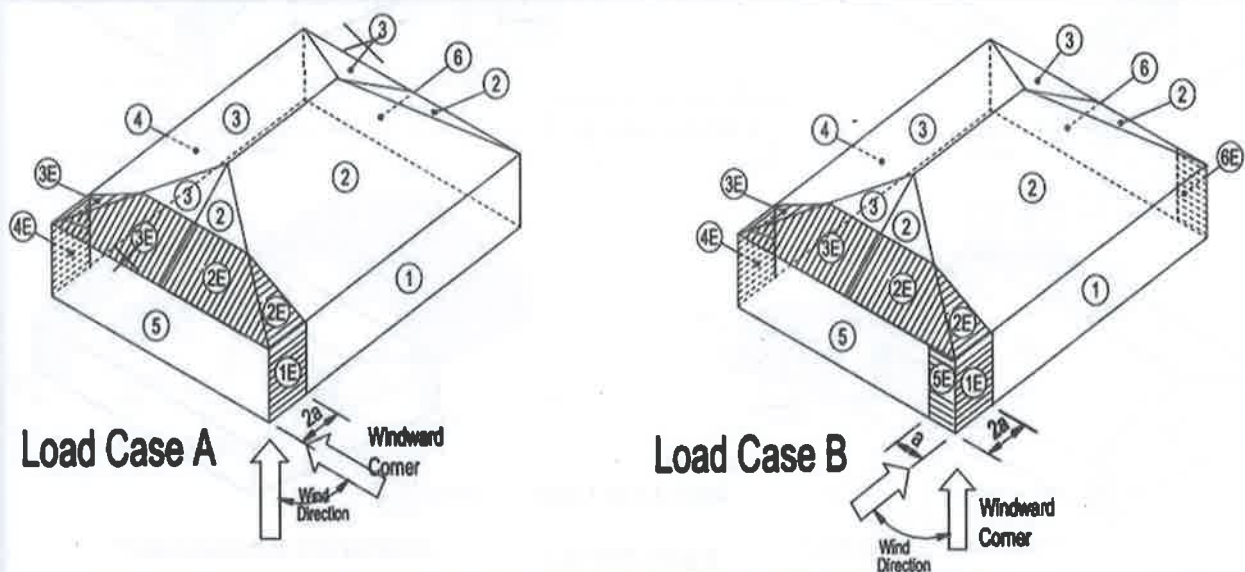
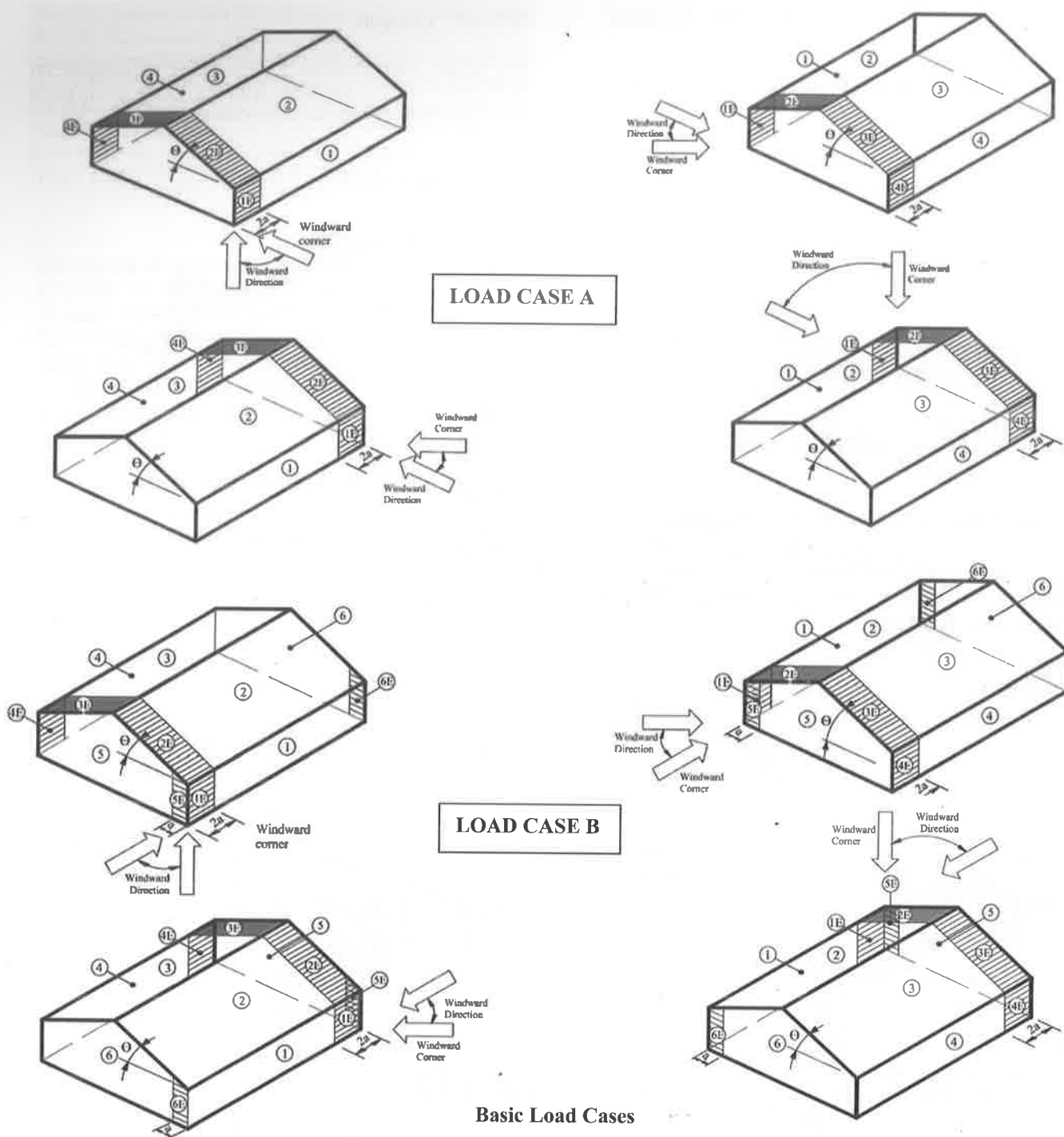


Figure C207C.4-2
Hip Roofed Low-Rise Buildings

Notes:

1. Adapt the loadings shown in Figure 207C.4-1 for hip roofed buildings as shown above. For a given hip roof pitch use the roof coefficients from the Case A table for both Load Case A and Load Case B.
2. The total horizontal shear shall not be less than that determined by neglecting the wind forces on roof surfaces.



Basic Load Cases

Figure 207C.4-1
Main Wind Force Resisting System – Part 1 External Pressure Coefficients (GC_{pf})
Enclosed, Partially Enclosed Buildings, $h \leq 18$ m Low-rise Walls and Roofs

Roof Angle θ (degrees)	LOAD CASE A							
	Building Surface							
	1	2	3	4	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	0.69	0.69	-0.48	-0.48

Roof Angle θ (degrees)	LOAD CASE B											
	Building Surface											
	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
0-90	-	-	-	-	0.4	-	-	-	-	-	0.61	-
	0.45	0.69	0.37	0.45	0	0.29	0.48	1.07	0.53	0.48		0.43

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. For values of θ other than those shown, linear interpolation is permitted.
3. The building must be designed for all wind directions using the 8 loading patterns shown. The load patterns are applied to each building corner in turn as the Windward Corner.
4. Combinations of external and internal pressures (see Table 207A.11-1) shall be evaluated as required to obtain the most severe loadings.
5. For the torsional load cases shown below, the pressures in zones designated with a "T" (1T, 2T, 3T, 4T, 5T, 6T) shall be 25% of the full design wind pressures (zones 1, 2, 3, 4, 5, 6).
Exception: One storey buildings with h less than or equal to 9 m, buildings two storeys or less framed with light frame construction, and buildings two storeys or less designed with flexible diaphragms need to be designed for the torsional load cases.
 Torsional loading shall apply to all eight basic load patterns using the figures below applied at each Windward Corner.
6. For purposes of designing a building's MWFRS, the total horizontal shear shall not be less than that determined by neglecting the wind forces on the roof.
Exception: This provision does not apply to buildings using moment frames for the MWFRS.
7. For flat roofs, use $\theta = 0^\circ$ and locate the zone 2/3 and zone 2E/3E boundary at the mid-width of the building.
8. The pressure coefficient (GC_{pf}), when negative in Zone 2 and 2E, shall be applied in Zone 2/2E for a distance from the edge of the roof equal 0.5 times the horizontal dimension of the building parallel to the direction of MWFRS being designed or 2.5 times the eave height at the windward wall, whichever is less; the remainder of Zone 2/2E extending to the ridge line shall use the pressure coefficient (GC_{pf}) for zone 3/3E.
9. Notation:
 a : 10% of least horizontal dimension or $.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m
 h : Mean roof height, in meters, except that eave height shall be used for $\theta \leq 10^\circ$.
 θ : Angle of plane of roof from horizontal, in degrees

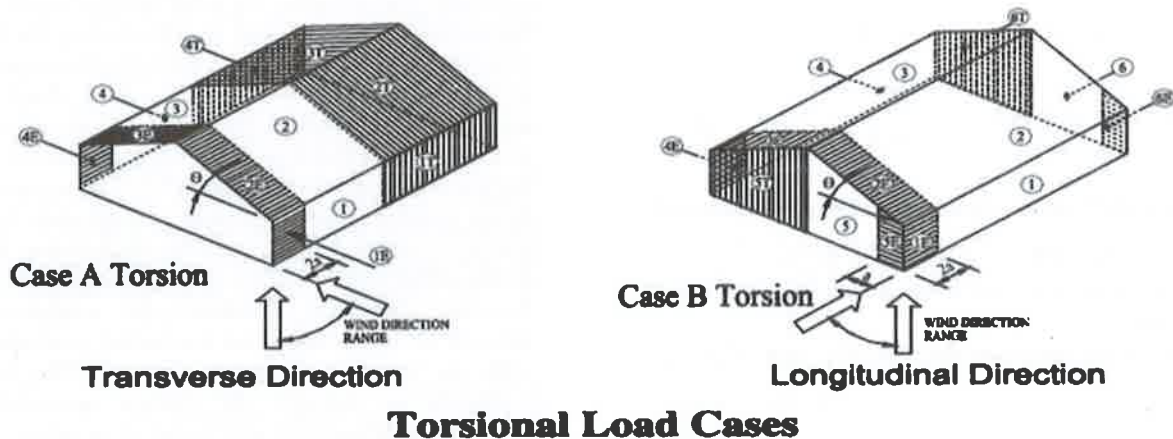


Figure 207C.4-1 (continued)
 Main Wind Force Resisting System – Part 1. External Pressure Coefficients (GC_{pf})
 Enclosed, Partially Enclosed Buildings, $h \leq 18\text{m}$ Low-rise Walls and Roofs

207C.4.3 Roof Overhangs

The positive external pressure on the bottom surface of windward roof overhangs shall be determined using $C_p = 0.7$ in combination with the top surface pressures determined using Figure 207C.4-1.

207C.4.4 Minimum Design Wind Loads

The wind load to be used in the design of the MWFRS for an enclosed or partially enclosed building shall not be less than 0.77 kN/m^2 multiplied by the wall area of the building and 0.38 kN/m^2 multiplied by the roof area of the building projected onto a vertical plane normal to the assumed wind direction.

Table 207C.3-1
Velocity Pressure Exposure Coefficients, K_h and K_z

Height above ground level, z m	Exposure		
	B	C	D
0-4.6	0.70	0.85	1.03
6.1	0.70	0.90	1.08
7.6	0.70	0.94	1.12
9.1	0.70	0.98	1.16
12.2	0.76	1.04	1.22
15.2	0.81	1.09	1.27
18	0.85	1.13	1.31

Notes:

1. The velocity pressure exposure coefficient K_z may be determined from the following formula:

For $4.57 \text{ m} \leq z \leq z_g$ For $z < 4.57 \text{ m}$

$$K_z = 2.01(z/z_g)^{2/\alpha} \quad K_z = 2.01(4.57/z_g)^{2/\alpha}$$

Note: z shall not be taken less than 9 m in exposure B.

2. α and z_g are tabulated in Table 207A.9-1.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposure categories are defined in Section 207A.7.

Commentary

This section specifies a minimum wind load to be applied horizontally on the entire vertical projection of the building as shown in Figure C207B.4-1. This load case is to be applied as a separate load case in addition to the normal load cases specified in other portions of this chapter.

Part 2: Enclosed Simple Diaphragm Low-Rise Buildings

Commentary:

This simplified approach of the Envelope Procedure is for the relatively common low-rise ($h \leq 18 \text{ m}$) regular shaped, simple diaphragm building case (see definitions for "simple diaphragm building" and "regular-shaped building") where pressures for the roof and walls can be selected directly from a table. Figure 207C.6-1 provides the design pressures for MWFRS for the specified conditions. Values are provided for enclosed buildings only ($(GC_{pi}) = \pm 0.18$).

Horizontal wall pressures are the net sum of the windward and leeward pressures on vertical projection of the wall. Horizontal roof pressures are the net sum of the windward and leeward pressures on vertical projection of the roof. Vertical roof pressures are the net sum of the external and internal pressures on the horizontal projection of the roof.

Note that for the MWFRS in a diaphragm building, the internal pressure cancels for loads on the walls and for the horizontal component of loads on the roof. This is true because when wind forces are transferred by horizontal diaphragms (e.g., floors and roofs) to the vertical elements of the MWFRS (e.g., shear walls, X-bracing, or moment frames), the collection of wind forces from windward and leeward sides of the building occurs in the horizontal diaphragms. Once transferred into the horizontal diaphragms by the vertically spanning wall systems, the wind forces become a net horizontal wind force that is delivered to the lateral force resisting elements of the MWFRS. There should be no structural separations in the diaphragms. Additionally, there should be no girts or other horizontal members that transmit significant wind loads directly to vertical frame members of the MWFRS in the direction under consideration. The equal and opposite internal pressures on the walls cancel each other in the horizontal diaphragm. This simplified approach of the Envelope Procedure combines the windward and leeward pressures into a net horizontal wind pressure, with the internal pressures canceled. The user is cautioned to consider the precise application of windward and leeward wall loads to members of the roof diaphragm where openings may exist and where particular members, such as drag struts, are designed. The design of the roof members of the MWFRS for vertical loads is influenced by internal pressures. The maximum uplift, which is controlled by Load Case B, is produced by a positive internal pressure. At a roof slope

of approximately 28° and above the windward roof pressure becomes positive and a negative internal pressure used in Load Case 2 in the table may produce a controlling case. From 25° to 45°, both positive and negative internal pressure cases (Load Cases 1 and 2, respectively) must be checked for the roof.

For the designer to use this method for the design of the MWFRS, the building must conform to all of the requirements listed in Section 207A.8.2; otherwise the Directional Procedure, Part 1 of the Envelope Procedure, or the Wind Tunnel Procedure must be used. This method is based on Part 1 of the Envelope Procedure, as shown in Figure 207C.4-1, for a specific group of buildings (simple diaphragm buildings). However, the torsional loading from Figure 207C.4-1 is deemed to be too complicated for a simplified method. The last requirement in Section 207C.6.2 prevents the use of this method for buildings with lateral systems that are sensitive to torsional wind loading.

Note 5 of Figure 207C.4-1 identifies several building types that are known to be insensitive to torsion and may therefore be designed using the provisions of Section 207C.6. Additionally, buildings whose lateral resistance in each principal direction is provided by two shear walls, braced frames, or moment frames that are spaced apart a distance not less than 75 percent of the width of the building measured normal to the orthogonal wind direction, and other building types and element arrangements described in Section 207B.6.1 or 207B.6.2 are also insensitive to torsion. This property could be demonstrated by designing the building using Part 1 of Section 207C, Figure 207C.4-1, and showing that the torsion load cases defined in Note 5 do not govern the design of any of the lateral resisting elements. Alternatively, it can be demonstrated within the context of Part 2 of Section 207C by defining torsion load cases based on the loads in Figure 207C.6-1 and reducing the pressures on one-half of the building by 75 percent, as described in Figure 207C.4-1, Note 5. If none of the lateral elements are governed by these torsion cases, then the building can be designed using Part 2 of Section 207C; otherwise the building must be designed using Part 1 of Section 207B or Part 1 of Section 207C.

Values are tabulated for Exposure B at $h = 9.0$ m, and $K_{zt} = 1.0$. Multiplying factors are provided for other exposures and heights. The following values have been used in preparation of the figures:

Exposure B

$$\begin{aligned} (GC_{pi}) &= \pm 0.18 \text{ (enclosed building)} \\ h &= 9.0 \text{ m} \\ K_d &= 0.85 \end{aligned}$$

$$K_z = 0.70$$

$$K_{zt} = 1.0$$

Pressure coefficients are from Figure 207C.4-1.

Wall elements resisting two or more simultaneous wind-induced structural actions (e.g., bending, uplift, or shear) should be designed for the interaction of the wind loads as part of the MWFRS. The horizontal loads in Figure 207C.6-1 are the sum of the windward and leeward pressures and are therefore not applicable as individual wall pressures for the interaction load cases. Design wind pressures, p_s , for zones A and C, should be multiplied by +0.85 for use on windward walls and by –0.70 for use on leeward walls (the plus sign signifies pressures acting toward the wall surface). For side walls, p_s for zone C multiplied by –0.65 should be used. These wall elements must also be checked for the various separately acting (not simultaneous) component and cladding load cases.

Main wind-force resisting roof members spanning at least from the eave to the ridge or supporting members spanning at least from eave to ridge are not required to be designed for the higher end zone loads. The interior zone loads should be applied. This is due to the enveloped nature of the loads for roof members.

207C.5 General Requirements

The steps required for the determination of MWFRS wind loads on enclosed simple diaphragm buildings are shown in Table 207C.5-1.

User Note:

Part 2 of Section 207C is a simplified method to determine the wind pressure on the MWFRS of enclosed simple diaphragm low-rise buildings having a flat, gable or hip roof. The wind pressures are obtained directly from a table and applied on horizontal and vertical projected surfaces of the building. This method is a simplification of the Envelope Procedure contained in Part 1 of Section 207C.

207C.5.1 Wind Load Parameters Specified in Section 207A

The following wind load parameters are specified in Section 207A:

- Basic Wind Speed V (Section 207A.5)
- Exposure category (Section 207A.7)

- Topographic factor K_{zt} (Section 207A.8)
- Enclosure classification (Section 207A.10)

Table 207C.5-1

Steps to Determine Wind Loads on MWFRS Simple Diaphragm Low-Rise Buildings

- | | |
|---------|--|
| Step 1: | Determine occupancy category of building or other structure, see Table 103-1 |
| Step 2: | Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C |
| Step 3: | Determine wind load parameters: <ul style="list-style-type: none"> ➤ Exposure category B, C or D, see Section 207A.7 ➤ Topographic factor, K_{zt}, see Section 207A.8 and Figure 207A.8-1 |
| Step 4: | Enter figure to determine wind pressures for $h = 9$ m, p_{s30} , see Figure 207C.6-1 |
| Step 5: | Enter figure to determine adjustment for building height and exposure, λ , see Figure 207C.6-1 |
| Step 6: | Determine adjusted wind pressures, p_s , see Equation 207C.6-1 |

207C.6 Wind Loads - Main Wind-Force Resisting System

207C.6.1 Scope

A building whose design wind loads are determined in accordance with this section shall meet all the conditions of Section 207C.6.2. If a building does not meet all of the conditions of Section 207C.6.2, then its MWFRS wind loads shall be determined by Part 1 of this chapter, by the Directional Procedure of Section 207B, or by the Wind Tunnel Procedure of Section 207F.

207C.6.2 Conditions

For the design of MWFRS the building shall comply with all of the following conditions:

1. The building is a simple diaphragm building as defined in Section 207A.2.
2. The building is a low-rise building as defined in Section 207A.2.
3. The building is enclosed as defined in Section 207A.2 and conforms to the wind-borne debris provisions of Section 207A.10.3.
4. The building is a regular-shaped building or structure as defined in Section 207A.2.
5. The building is not classified as a flexible building as defined in Section 207A.2.
6. The building does not have response characteristics making it subject to across wind loading, vortex shedding, instability due to galloping or flutter; and it does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
7. The building has an approximately symmetrical cross-section in each direction with either a flat roof or a gable or hip roof with $\theta \leq 45^\circ$.
8. The building is exempted from torsional load cases as indicated in Note 5 of Figure 207C.4-1, or the torsional load cases defined in Note 5 do not control the design of any of the MWFRS of the building.

207C.6.3 Design Wind Loads

Simplified design wind pressures, p_s , for the MWFRS of low-rise simple diaphragm buildings represent the net pressures (sum of internal and external) to be applied to the horizontal and vertical projections of building surfaces as shown in Figure 207C.6-1. For the horizontal pressures (Zones A, B, C, D), p_s is the combination of the windward and leeward net pressures. p_s shall be determined by the following equation:

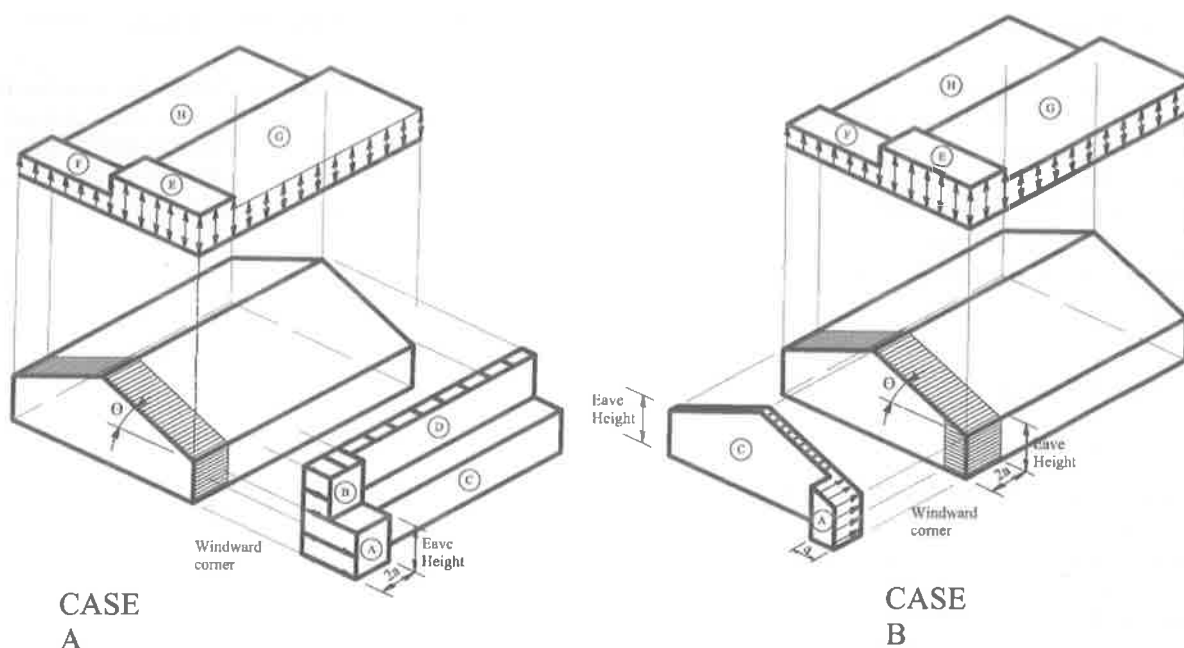
$$p_s = \lambda K_{zt} p_{s30} \quad (207C.6-1)$$

where

- λ adjustment factor for building height and exposure from Figure 207C.6-1
- K_{zt} topographic factor as defined in Section 207A.8 evaluated at mean roof height, h
- p_{s30} simplified design wind pressure for Exposure B, at $h = 9$ m from Figure 207C.6-1

207C.6.4 Minimum Design Wind Loads

The load effects of the design wind pressures from Section 207C.6.3 shall not be less than a minimum load defined by assuming the pressures, p_s , for zones A and C equal to +766 Pa, Zones B and D equal to +383 Pa, while assuming p_s for Zones E, F, G, and H are equal to 0.0 Pa.



Notes:

1. Pressures shown are applied to the horizontal and vertical projections, for exposure B, at $h = 9$ m. Adjust to other exposures and height with adjustment factor λ .
2. The load patterns shown shall be applied to each corner of the building in turn as the reference corner. (See Figure 207C.4-1)
3. For Case B use $\theta = 0^\circ$.
4. Load cases 1 and 2 must be checked for $25^\circ < \theta \leq 45^\circ$. Load case 2 at 25° is provided only for interpolation between 25° and 30° .
5. Plus and minus signs signify pressures acting toward and away from the projected surfaces, respectively.
6. For roof slopes other than those shown, linear interpolation is permitted.
7. The total horizontal load shall not be less than that determined by assuming $p_s = 0$ in zones B and D.
8. Where zone E or G falls on a roof overhang on the windward side of the building, use E_{OH} and G_{OH} for the pressure on the horizontal projection of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied.
9. Notation:
 - α : 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m
 - h : Mean roof height, in meters, except that eave height shall be used for roof angles $< 10^\circ$.
 - θ : Angle of plane of roof from horizontal, in degrees.

Figure 207C.6-1
Main Wind Force Resisting System – Method 2. Design Wind Pressure
Enclosed Buildings $h \leq 18$ m Walls and Roofs

Basic Wind Speed (kph)	Roof Angle (degrees)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	EOH	GOH
150	0 to 5°	1	0.66	-0.34	0.44	-0.2	-0.79	-0.45	-0.55	-0.35	-1.11	-0.87
	10°	1	0.74	-0.31	0.49	-0.18	-0.79	-0.48	-0.55	-0.37	-1.11	-0.87
	15°	1	0.83	-0.27	0.55	-0.16	-0.79	-0.52	-0.55	-0.39	-1.11	-0.87
	20°	1	0.91	-0.24	0.61	-0.13	-0.79	-0.55	-0.55	-0.42	-1.11	-0.87
	25°	1	0.83	0.13	0.6	0.14	-0.37	-0.5	-0.27	-0.41	-0.68	-0.58
		2	0	0	0	0	-0.14	-0.27	-0.04	-0.18	0	0
	30 to 45°	1	0.74	0.51	0.59	0.41	0.06	-0.45	0.02	-0.39	-0.26	-0.3
		2	0.74	0.51	0.59	0.41	0.29	-0.22	0.25	-0.16	-0.26	-0.3
200	0 to 5°	1	1.17	-0.61	0.78	-0.36	-1.41	-0.8	-0.98	-0.62	-1.97	-1.54
	10°	1	1.32	-0.55	0.88	-0.32	-1.41	-0.86	-0.98	-0.67	-1.97	-1.54
	15°	1	1.48	-0.48	0.98	-0.28	-1.41	-0.92	-0.98	-0.7	-1.97	-1.54
	20°	1	1.62	-0.43	1.08	-0.23	-1.41	-0.98	-0.98	-0.74	-1.97	-1.54
	25°	1	1.48	0.23	1.07	0.25	-0.65	-0.89	-0.47	-0.72	-1.22	-1.03
		2	0	0	0	0	-0.25	-0.48	-0.07	-0.32	0	0
	30 to 45°	1	1.32	0.9	1.05	0.72	0.1	-0.8	0.03	-0.68	-0.46	-0.53
		2	1.32	0.9	1.05	0.72	0.51	-0.39	0.44	-0.28	-0.46	-0.53
250	0 to 5°	1	1.83	-0.95	1.22	-0.57	-2.2	-1.25	-1.53	-0.97	-3.08	-2.41
	10°	1	2.06	-0.86	1.37	-0.49	-2.2	-1.34	-1.53	-1.04	-3.08	-2.41
	15°	1	2.31	-0.76	1.53	-0.44	-2.2	-1.44	-1.53	-1.09	-3.08	-2.41
	20°	1	2.53	-0.67	1.69	-0.37	-2.2	-1.53	-1.53	-1.16	-3.08	-2.41
	25°	1	2.31	0.37	1.67	0.39	-1.02	-1.39	-0.74	-1.13	-1.9	-1.62
		2	0	0	0	0	-0.39	-0.76	-0.11	-0.49	0	0
	30 to 45°	1	2.06	1.41	1.64	1.13	0.16	-1.25	0.05	-1.07	-0.72	-0.83
		2	2.06	1.41	1.64	1.13	0.79	-0.62	0.68	-0.44	-0.72	-0.83

Figure 207C.6-1

Main Wind Force Resisting System – Method 2 Design Wind Pressure
 Enclosed Buildings $h \leq 18$ m Walls and Roofs, Simplified Design Wind Pressure, $p_{s9.0}$ (Pa)
 (Exposure B at $h = 9.0$ m with $I = 1.0$)

Basic Wind Speed (kph)	Roof Angle (degrees)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	EOH	GOH
300	0 to 5°	1	2.63	-1.37	1.75	-0.81	-3.17	-1.8	-2.2	-1.39	-4.44	-3.47
	10°	1	2.97	-1.24	1.98	-0.71	-3.17	-1.93	-2.2	-1.49	-4.44	-3.47
	15°	1	3.32	-1.09	2.2	-0.63	-3.17	-2.08	-2.2	-1.57	-4.44	-3.47
	20°	1	3.65	-0.96	2.43	-0.53	-3.17	-2.2	-2.2	-1.67	-4.44	-3.47
	25°	1	3.32	0.53	2.41	0.56	-1.47	-2	-1.06	-1.62	-2.74	-2.33
		2	0	0	0	0	-0.56	-1.09	-0.15	-0.71	0	0
	30 to 45°	1	2.97	2.03	2.36	1.62	0.23	-1.8	0.08	-1.55	-1.04	-1.19
		2	2.97	2.03	2.36	1.62	1.14	-0.89	0.99	-0.63	-1.04	-1.19
350	0 to 5°	1	3.59	-1.86	2.38	-1.11	-4.32	-2.45	-3	-1.9	-6.04	-4.73
	10°	1	4.04	-1.69	2.69	-0.97	-4.32	-2.62	-3	-2.04	-6.04	-4.73
	15°	1	4.52	-1.48	3	-0.86	-4.32	-2.83	-3	-2.14	-6.04	-4.73
	20°	1	4.97	-1.31	3.31	-0.72	-4.32	-3	-3	-2.28	-6.04	-4.73
	25°	1	4.52	0.72	3.28	0.76	-2	-2.73	-1.45	-2.21	-3.73	-3.18
		2	0	0	0	0	-0.76	-1.48	-0.21	-0.97	0	0
	30 to 45°	1	4.04	2.76	3.21	2.21	0.31	-2.45	0.11	-2.1	-1.41	-1.62
		2	4.04	2.76	3.21	2.21	1.55	-1.21	1.35	-0.86	-1.41	-1.62

Adjustment Factor for Building Height and Exposure, λ

Mean Roof Height (m)	Exposure		
	B	C	D
0-5	1.0	1.21	1.47
6	1.0	1.29	1.55
8	1.0	1.35	1.61
9	1.0	1.40	1.66
11	1.05	1.45	1.70
12	1.09	1.49	1.74
14	1.12	1.53	1.78
15	1.16	1.56	1.81
17	1.19	1.59	1.84
18	1.22	1.62	1.87

Figure 207C.6-1 (continued)

Main Wind Force Resisting System – Method 2 Design Wind Pressure
 Enclosed Buildings $h \leq 18$ m Walls and Roofs, Simplified Design Wind Pressure, $p_{S9.0}$ (Pa)
 (Exposure B at $h = 9.0$ m with $I = 1.0$)

207D Wind Loads on Other Structures and Building Appurtenances - MWFRS

207D.1 Scope

207D.1.1 Structure Types

This section applies to the determination of wind loads on building appurtenances (such as rooftop structures and rooftop equipment) and other structures of all heights (such as solid freestanding walls and freestanding solid signs, chimneys, tanks, open signs, lattice frameworks, and trussed towers) using the Directional Procedure.

The steps required for the determination of wind loads on building appurtenances and other structures are shown in Table 207D.1-1.

User Note:

Use Section 207D to determine wind pressures on the MWFRS of solid freestanding walls, freestanding solid signs, chimneys, tanks, open signs, lattice frameworks and trussed towers. Wind loads on rooftop structures and equipment may be determined from the provisions of this chapter. The wind pressures are calculated using specific equations based upon the Directional Procedure.

207D.1.2 Conditions

A structure whose design wind loads are determined in accordance with this section shall comply with all of the following conditions:

1. The structure is a regular-shaped structure as defined in Section 207A.2.
2. The structure does not have response characteristics making it subject to across-wind loading, vortex shedding, or instability due to galloping or flutter; or it does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

207D.1.3 Limitations

The provisions of this chapter take into consideration the load magnification effect caused by gusts in resonance with along-wind vibrations of flexible structures. Structures not meeting the requirements of Section 207D.1.2, or having unusual shapes or response characteristics, shall be

designed using recognized literature documenting such wind load effects or shall use the Wind Tunnel Procedure specified in Section 207F.

207D.1.4 Shielding

There shall be no reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

207D.2 General Requirements

207D.2.1 Wind Load Parameters Specified in Section 207A

The following wind load parameters shall be determined in accordance with Section 207A:

- Basic Wind Speed V (Section 207A.5)
- Wind directionality Factor K_d (Section 207A.6)
- Exposure category (Section 207A.7)
- Topographic factor K_{zt} (Section 207A.8)
- Enclosure classification (Section 207A.10)

207D.3 Velocity Pressure

207D.3.1 Velocity Pressure Exposure Coefficient

Based on the exposure category determined in Section 207A.7.3, a velocity pressure exposure coefficient K_z or K_h , as applicable, shall be determined from Table 207D.3-1.

For a site located in a transition zone between exposure categories that is near to a change in ground surface roughness, intermediate values of K_z or K_h , between those shown in Table 207D.3-1, are permitted, provided that they are determined by a rational analysis method defined in the recognized literature.

Commentary:

See commentary, Section C207B.3.1.

207D.3.2 Velocity Pressure

Velocity pressure, q_z , evaluated at height z shall be calculated by the following equation:

$$q_z = 0.613K_zK_{zt}K_dV^2 \text{ (N/m}^2\text{); } V \text{ in m/s} \quad (207D.3-1)$$

where

- K_d = wind directionality factor, see Section 207A.6
- K_z = velocity pressure exposure coefficient, see Section 207D.3.1
- K_{zt} = topographic factor defined, see Section 207A.8.2
- V = basic wind speed, see Section 207A.5
- q_h = velocity pressure calculated using Equation 207D.3-1 at height h

The numerical coefficient 0.613 shall be used except where sufficient climatic data are available to justify the selection of a different value of this factor for a design application.

Table 207D.1-1
Steps to Determine Wind Loads on MWFRS Rooftop
Equipment and Other Structures

- | | |
|---------|---|
| Step 1: | Determine occupancy category of building or other structure, see Table 103-1 |
| Step 2: | Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C |
| Step 3: | Determine wind load parameters: <ul style="list-style-type: none"> ➤ Wind directionality factor, K_d, see Section 207A.6 and Table 207A.6-1 ➤ Exposure category B, C or D, see Section 207A.7 ➤ Topographic factor, K_{zt}, see Section 207A.8 and Figure 207A.8-1 ➤ Gust Effect Factor, G, see Section 207A.9 |
| Step 4: | Determine velocity pressure exposure coefficient, K_z or K_h , see Table 207D.2-1 |
| Step 5: | Determine velocity pressure q_z or q_h , see Equation 207D.3-1 |
| Step 6: | Determine force coefficient, C_f : <ul style="list-style-type: none"> ➤ Solid freestanding signs or solid freestanding walls, Figure 207D.4-1 ➤ Chimneys, tanks, rooftop equipment Figure 207D.5-1 ➤ Open signs, lattice frameworks Figure 207D.5-2 ➤ Trussed towers Figure 207D.4-3 |
| Step 7: | Calculate wind force, F : <ul style="list-style-type: none"> ➤ Equation 207D.4-1 for signs and walls ➤ Equation 207D.6-1 and Equation 207D.6-2 for rooftop structures and equipment ➤ Equation 207D.5-1 for other structures |

Commentary:

See commentary, Section C207B.3.2.

Figure 207D.4-1. The force coefficients for solid freestanding walls and signs in Figure 207D.4-1 date back to ANSI A58.1-1972. It was shown by Letchford (2001) that these data originated from wind tunnel studies performed by Flachsbart in the early 1930s in smooth uniform flow. The current values in Figure 207D.4-1 are based on the results of boundary layer wind tunnel studies (Letchford 1985, 2001, Holmes 1986, Letchford and Holmes 1994, Ginger et al. 1998a and 1998b, and Letchford and Robertson 1999).

A surface curve fit to Letchford's (2001) and Holmes's (1986) area averaged mean net pressure coefficient data (equivalent to mean force coefficients in this case) is given by the following equation

$$C_f = \left\{ \begin{array}{l} 1.563 + 0.0042 \ln(x) - 0.06148y \\ + 0.009011[\ln(x)]^2 - 0.2603y^2 \\ - 0.08393y[\ln(x)] \end{array} \right\} / 0.8$$

where $x = B/s$ and $y = s/h$

The 0.85 term in the denominator modifies the wind tunnel-derived force coefficients into a format where the gust effect factor as defined in Section 207A.9 can be used.

Force coefficients for Cases A and B were generated from the preceding equation, then rounded off to the nearest 0.05. That equation is only valid within the range of B/s and s/h ratios given in the figure for Case A and B.

Of all the pertinent studies, only Letchford (2001) specifically addressed eccentricity (i.e., Case B). Letchford reported that his data provided a reasonable match to Cook's (1990) recommendation for using an eccentricity of 0.25 times the average width of the sign. However, the data were too limited in scope to justify changing the existing eccentricity value of 0.2 times the average width of the sign, which is also used in the latest Australian / New Zealand Standard (Standards Australia 2002).

Case C was added to account for the higher pressures observed in both wind tunnel (Letchford 1985, 2001, Holmes 1986, Letchford and Holmes 1994, Ginger et al. 1998a and 1998b, and Letchford and Robertson 1999) and full-scale studies (Robertson et al. 1997) near the windward edge of a freestanding wall or sign for oblique wind directions. Linear regression equations were fit to

the local mean net pressure coefficient data (for wind direction 45°) from the referenced wind tunnel studies to generate force coefficients for square regions starting at the windward edge. Pressures near this edge increase significantly as the length of the structure increases. No data were available on the spatial distribution of pressures for structures with low aspect ratios ($B/s < 2$).

The sample illustration for Case C at the top of Figure 207D.4-1 is for a sign with an aspect ratio $B/s = 4$. For signs of differing B/s ratios, the number of regions is equal to the number of force coefficient entries located below each B/s column heading.

For oblique wind directions (Case C), increased force coefficients have been observed on aboveground signs compared to the same aspect ratio walls on ground (Letchford 1985, 2001 and Ginger et al. 1998a). The ratio of force coefficients between above-ground and on-ground signs (i.e., $s/h = 0.8$ and 1.0, respectively) is 1.25, which is the same ratio used in the Australian / New Zealand Standard (Standards Australia 2002). Note 5 of Figure 207D.4-1 provides for linear interpolation between these two cases.

For walls and signs on the ground ($s/h = 1$), the mean vertical center of pressure ranged from $0.5h$ to $0.6h$ (Holmes 1986, Letchford 1989, Letchford and Holmes 1994, Robertson et al. 1995, 1996, and Ginger et al. 1998a) with $0.55h$ being the average value. For above-ground walls and signs, the geometric center best represents the expected vertical center of pressure.

The reduction in C_f due to porosity (Note 2) follows a recommendation (Letchford 2001). Both wind tunnel and full-scale data have shown that return corners significantly reduce the net pressures in the region near the windward edge of the wall or sign (Letchford and Robertson 1999).

207D.4 Design Wind Loads - Solid Freestanding Walls and Solid Signs

207D.4.1 Solid Freestanding Walls and Solid Freestanding Signs

The design wind force for solid freestanding walls and solid freestanding signs shall be determined by the following formula:

$$F = q_h G C_f A_s \text{ (N)} \quad (207D.4-1)$$

where

- q_h = velocity pressure evaluated at height h (defined in Figure 207D.4-1) as determined in accordance with Section 207D.3.2
- G = gust-effect factor from Section 207A.9
- C_f = net force coefficient from Figure 207D.4-1
- A_s = the gross area of the solid freestanding wall or freestanding solid sign, m^2

207D.4.2 Solid Attached Signs

The design wind pressure on a solid sign attached to the wall of a building, where the plane of the sign is parallel to and in contact with the plane of the wall, and the sign does not extend beyond the side or top edges of the wall, shall be determined using procedures for wind pressures on walls in accordance with Section 207E, and setting the internal pressure coefficient (GC_{pi}) equal to 0.

This procedure shall also be applicable to solid signs attached to but not in direct contact with the wall, provided the gap between the sign and wall is no more than 0.9 m and the edge of the sign is at least 0.9 m in from free edges of the wall, i.e., side and top edges and bottom edges of elevated walls.

Commentary:

Signs attached to walls and subject to the geometric limitations of Section 207D.4.2 should experience wind pressures approximately equal to the external pressures on the wall to which they are attached. The dimension requirements for signs supported by frameworks, where there is a small gap between the sign and the wall, are based on the collective judgment of the committee.

Figures 207D.5-1, 207D.5-2 and 207D.5-3. With the exception of Figure 207D.5-3, the pressure and force coefficient values in these tables are unchanged from ANSI A58.1-1972. The coefficients specified in these tables are based on wind-tunnel tests conducted under conditions of uniform flow and low turbulence, and their validity in

turbulent boundary-layer flows has yet to be completely established. Additional pressure coefficients for conditions not specified herein may be found in two references (SIA 1956 and ASCE 1961).

With regard to Figure 207D.5-3, the force coefficients are a refinement of the coefficients specified in ANSI A58.1-1982 and in ASCE 7-93. The force coefficients specified are offered as a simplified procedure that may be used for trussed towers and are consistent with force coefficients given in ANSI/EIA/TIA-222-E-1991, Structural Standards for Steel Antenna Towers and Antenna Supporting Structures, and force coefficients recommended by Working Group No. 4 (Recommendations for Guyed Masts), International Association for Shell and Spatial Structures (1981).

It is not the intent of this code to exclude the use of other recognized literature for the design of special structures, such as transmission and telecommunications towers. Recommendations for wind loads on tower guys are not provided as in previous editions of the code. Recognized literature should be referenced for the design of these special structures as is noted in Section 207D.1.3. For the design of flagpoles, see ANSI/NAAMM FP1001-97, 4th Ed., Guide Specifications for Design of Metal Flagpoles.

207D.5 Design Wind Loads—Other Structures

The design wind force for other structures (chimneys, tanks, rooftop equipment for $h > 60^\circ$, and similar structures, open signs, lattice frameworks, and trussed towers) shall be determined by the following equation:

$$F = q_z G C_f A_f \text{ (N)} \quad (207D.5-1)$$

where

- q_z = velocity pressure evaluated at height z as defined in Section 207D.3, of the centroid of area A_f
- G = gust-effect factor from Section 207A.9
- C_f = force coefficients from Figures 207D.5-1 through 207D.5-3
- A_s = projected area normal to the wind except where C_f is specified for the actual surface area, m^2

207D.5.1 Rooftop Structures and Equipment for Buildings with $h \leq 18$ m

The lateral force F_h on rooftop structures and equipment located on buildings with a mean roof height $h \leq 18$ m shall be determined from Equation 207D.5-2.

$$F_h = q_h(GC_r)A_f \text{ (N)} \quad (207D.5-2)$$

where

- (GC_r) = 1.9 for rooftop structures and equipment with A_f less than $(0.1Bh)$. (GC_r) shall be permitted to be reduced linearly from 1.9 to 1.0 as the value of A_f is increased from $(0.1Bh)$ to (Bh)
- q_h = velocity pressure evaluated at mean roof height of the building
- A_f = vertical projected area of the rooftop structure or equipment on a plane normal to the direction of wind, m^2

The vertical uplift force, F_v , on rooftop structures and equipment shall be determined from Equation 207D.5-3.

$$F_v = q_h(GC_r)A_r \text{ (N)} \quad (207D.5-3)$$

Where

- (GC_r) = 1.5 for rooftop structures and equipment with A_r less than $(0.1BL)$. (GC_r) shall be permitted to be reduced linearly from 1.5 to 1.0 as the value of A_r is increased from $(0.1BL)$ to (BL)
- q_h = velocity pressure evaluated at mean roof height of the building
- A_r = horizontal projected area of rooftop structure or equipment, m^2

Commentary:

This code requires the use of Figure 207D.5-1 for the determination of the wind force on small structures and equipment located on a rooftop. Because of the small size of the structures in comparison to the building, it is expected that the wind force will be higher than predicted by Equation 207D.6-1 due to higher correlation of pressures across the structure surface, higher turbulence on the building roof, and accelerated wind speed on the roof.

A limited amount of research is available to provide better guidance for the increased force (Hosoya et al. 2001 and Kopp and Traczuk 2008). Based on this research, the force of Equation 207D.6-1 should be increased for units with

areas that are relatively small with respect to that of the buildings they are on. Because GC_r is expected to approach 1.0 as A_f or A_r approaches that of the building (Bh or BL), a linear interpolation is included as a way to avoid a step function in load if the designer wants to treat other sizes. The research in Hosoya et al (2001) only treated one value of A_f ($0.04Bh$). The research in Kopp and Traczuk (2008) treated values of $A_f = 0.02Bh$ and $0.03Bh$, and values of $A_r = 0.0067BL$.

In both cases the research also showed high uplifts on the top of rooftop. Hence uplift load should also be considered by the designer and is addressed in Section 207D.6.

207D.6 Parapets

Wind loads on parapets are specified in Section 207B.4.5 for buildings of all heights designed using the Directional Procedure and in Section 207C.4.2 for low-rise buildings designed using the Envelope Procedure.

Commentary:

Prior to the 2002 edition of the standard, no provisions for the design of parapets had been included due to the lack of direct research. In the 2002 edition of this standard, a rational method was added based on the committee's collective experience, intuition, and judgment. In the 2005 edition, the parapet provisions were updated as a result of research performed at the University of Western Ontario (Mans et al. 2000, 2001) and at Concordia University (Stathopoulos et al. 2002a, 2002b).

Wind pressures on a parapet are a combination of wall and roof pressures, depending on the location of the parapet and the direction of the wind (Figure C207D.7-1). A windward parapet will experience the positive wall pressure on its front surface (exterior side of the building) and the negative roof edge zone pressure on its back surface (roof side). This behavior is based on the concept that the zone of suction caused by the wind stream separation at the roof eave moves up to the top of the parapet when one is present. Thus the same suction that acts on the roof edge will also act on the back of the parapet.

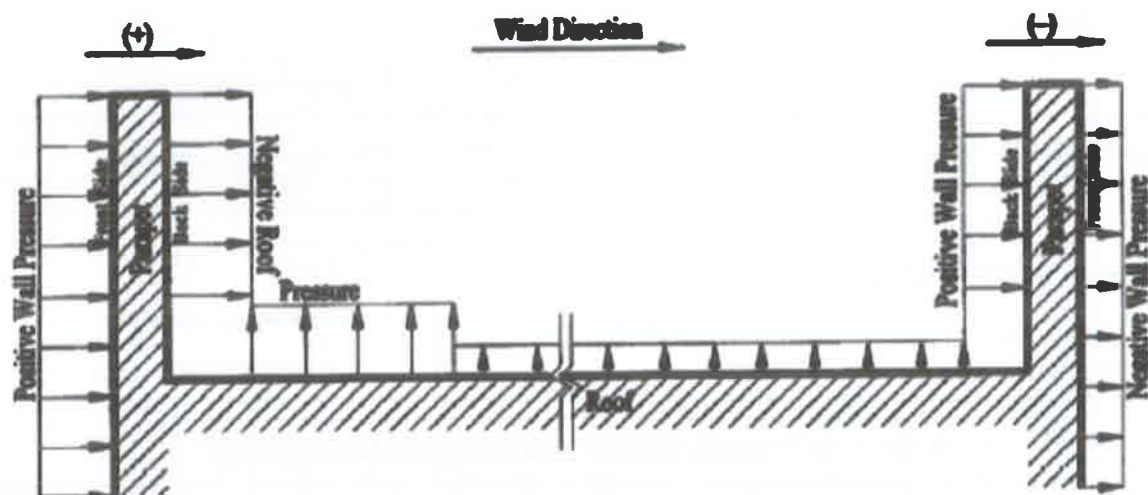
The leeward parapet will experience a positive wall pressure on its back surface (roof side) and a negative wall pressure on its front surface (exterior side of the building). There should be no reduction in the positive wall pressure to the leeward parapet due to shielding by the windward parapet because, typically, they are too far apart to experience this effect. Because all parapets would be designed for all wind directions, each parapet would in

turn be the windward and leeward parapet and, therefore, must be designed for both sets of pressures.

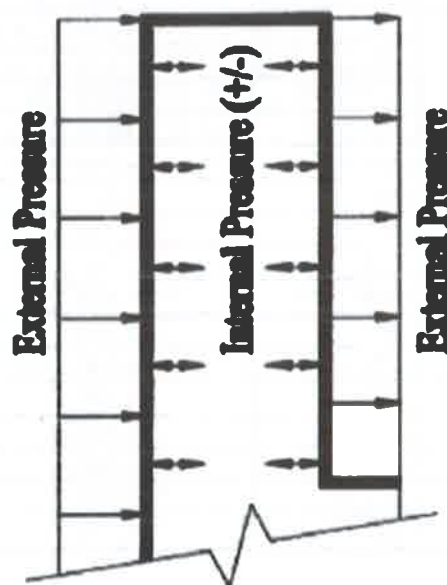
For the design of the MWFRS, the pressures used describe the contribution of the parapet to the overall wind loads on that system. For simplicity, the front and back pressures on the parapet have been combined into one coefficient for MWFRS design. The designer should not typically need the separate front and back pressures for MWFRS design. The internal pressures inside the parapet cancel out in the determination of the combined coefficient. The summation of these external and internal, front and back pressure coefficients is a new term GC_{pn} , the Combined Net Pressure Coefficient for a parapet.

For the design of the components and cladding, a similar approach was used. However, it is not possible to simplify the coefficients due to the increased complexity of the components and cladding pressure coefficients. In addition, the front and back pressures are not combined because the designer may be designing separate elements on each face of the parapet. The internal pressure is required to determine the net pressures on the windward and leeward surfaces of the parapet. The provisions guide the designer to the correct GC_p and velocity pressure to use for each surface, as illustrated in Figure C207D.7-1. Interior walls that protrude through the roof, such as party walls and fire walls, should be designed as windward parapets for both MWFRS and components and cladding.

The internal pressure that may be present inside a parapet is highly dependent on the porosity of the parapet envelope. In other words, it depends on the likelihood of the wall surface materials to leak air pressure into the internal cavities of the parapet. For solid parapets, such as concrete or masonry, the internal pressure is zero because there is no internal cavity. Certain wall materials may be impervious to air leakage, and as such have little or no internal pressure or suction, so using the value of GC_{pi} for an enclosed building may be appropriate. However, certain materials and systems used to construct parapets containing cavities are more porous, thus justifying the use of the GC_{pi} values for partially enclosed buildings, or higher. Another factor in the internal pressure determination is whether the parapet cavity connects to the internal space of the building, allowing the building's internal pressure to propagate into the parapet. Selection of the appropriate internal pressure coefficient is left to the judgment of the design professional.



Methodology used to Develop External Parapet Pressures
 (Main Wind Force Resisting Systems and Components and Cladding)



External and Internal Parapet Pressures
 (Components and Cladding Only)

Figure C207D.7-1
 Design Wind Pressures on Parapets

207D.7 Roof Overhangs

Wind loads on roof overhangs are specified in Section 207B.4.4 for buildings of all heights designed using the Directional Procedure and in Section 207C.4.3 for low-rise buildings designed using the Envelope Procedure.

207D.8 Minimum Design Wind Loading

The design wind force for other structures shall be not less than 0.77 kN/m^2 multiplied by the area A_f .

Commentary:

This section specifies a minimum wind load to be applied horizontally on the entire vertical projection of the building or other structure, as shown in Figure C207B.4-1. This load case is to be applied as a separate load case in addition to the normal load cases specified in other portions of this chapter.

Table 207D.3-1 Velocity Pressure Exposure Coefficients, K_h and K_z

Height above ground level, z (meters)	Exposure		
	B	C	D
0 - 4.5	0.572	0.846	1.027
6.0	0.621	0.899	1.080
7.5	0.662	0.942	1.123
9.0	0.697	0.979	1.159
12.0	0.757	1.040	1.218
15.0	0.807	1.090	1.267
18.0	0.850	1.133	1.307
21.0	0.888	1.170	1.343
24.0	0.923	1.204	1.375
27.0	0.955	1.234	1.403
30.0	0.984	1.261	1.429
36.0	1.036	1.311	1.475
42.0	1.083	1.354	1.515
48.0	1.125	1.393	1.551
54.0	1.164	1.428	1.583
60.0	1.199	1.460	1.612
75.0	1.278	1.530	1.676
90.0	1.347	1.590	1.730
105.0	1.407	1.642	1.777
120.0	1.462	1.689	1.819
135.0	1.512	1.731	1.856
150.0	1.558	1.770	1.891

Notes:

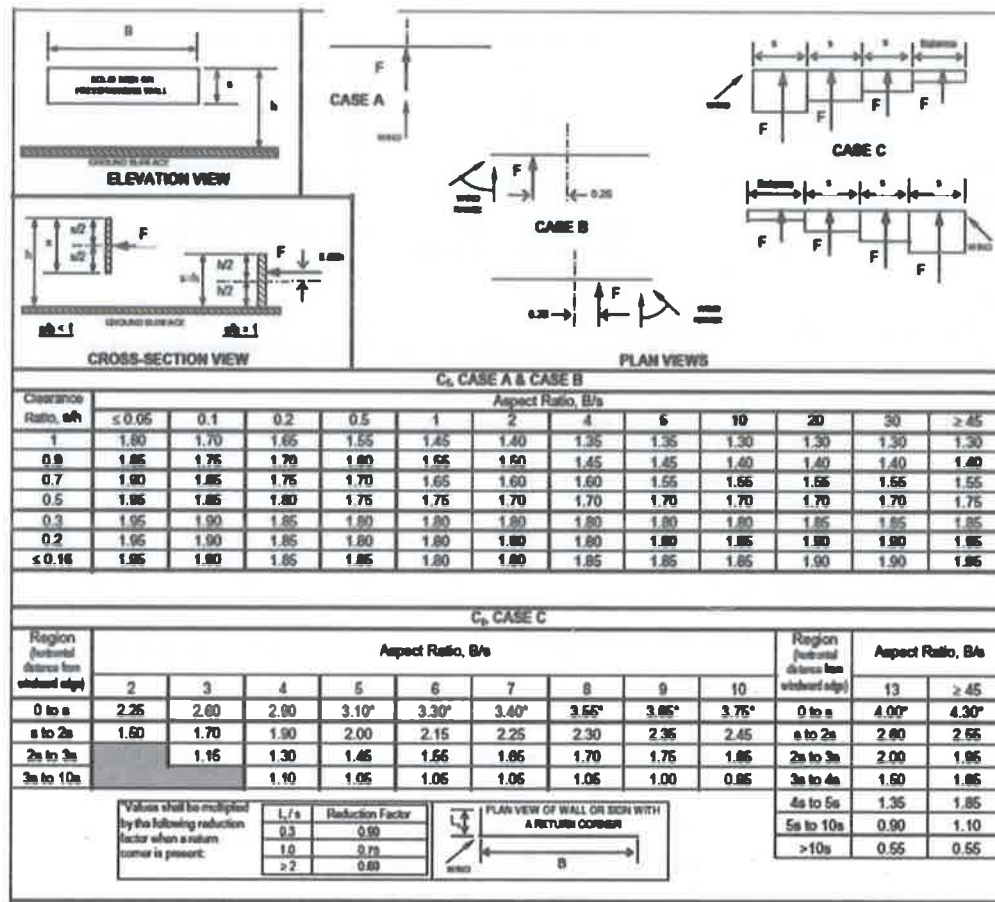
1. The velocity pressure exposure coefficient K_z may be determined from the following formula:

For $4.5 \text{ m} \leq z \leq z_g$ For $z < 4.5 \text{ m}$

$$K_z = 2.01(z/z_g)^{2/\alpha}$$

$$K_z = 2.01(4.57/z_g)^{2/\alpha}$$

2. The constants α and z_g are tabulated in Table 207A.9-1.
3. Linear interpolation for intermediate values of height z is permitted.
4. Exposure categories are defined in Section 207A.7.



Notes:

- The term "signs" in notes below also applies to "freestanding walls".
- Signs with openings comprising less than 30% of the gross area are classified as solid signs. Force coefficients for solid signs with openings shall be permitted to be multiplied by the reduction factor $(1 - (1 - \epsilon)^{1.5})$.
- To allow for both normal and oblique wind directions, the following cases shall be considered:
 - For $s/h < 1$:
 - CASE A: resultant force acts normal to the face of the sign through the geometric center.
 - CASE B: resultant force acts normal to the face of the sign at a distance from the geometric center toward the windward edge equal to 0.2 times the average width of the sign.
 - For $B/s \geq 2$, CASE C must also be considered:
 - CASE C: resultant forces act normal to the face of the sign through the geometric centers of each region.
- For $s/h = 1$:
 - The same cases as above except that the vertical locations of the resultant forces occur at a distance above the geometric center equal to 0.05 times the average height of the sign.
- For CASE C where $s/h > 0.8$, force coefficients shall be multiplied by the reduction factor $(1.8 - s/h)$.
- Linear interpolation is permitted for values of s/h , B/s and L_r/s other than shown.
- Notation:
 - B : horizontal dimension of sign, m;
 - h : height of the sign, m
 - s : vertical dimension of the sign, m;
 - ϵ : ratio of solid area to gross area;
 - L_r : horizontal dimension of return corner, m

Figure 207D.4-1
Design Wind Loads Force Coefficients (C_f) Other Structures
All Heights of Solid Freestanding Walls & Solid Freestanding Signs

Cross-Section	Type of Surface	h/D		
		1	7	25
Square (wind normal to face)	All	1.3	1.4	2.0
Square (wind along diagonal)	All	1.0	1.1	1.5
Hexagonal or Octagonal	All	1.0	1.2	1.4
Round ($D\sqrt{q_z} > 2.5$) ($D\sqrt{q_z} > 5.3$, D in m, q_z in N/m^2)	Moderately Smooth	0.5	0.6	0.7
	Rough ($D'/D = 0.02$)	0.7	0.8	0.9
	Very Rough ($D'/D = 0.08$)	0.8	1.0	1.2
Round ($D\sqrt{q_z} \leq 2.5$) ($D\sqrt{q_z} \leq 5.3$, D in m, q_z in N/m^2)	All	0.7	0.8	1.2

Notes:

- The design wind force shall be calculated based on the area of the structure projected on a plane normal to the wind direction. The force shall be assumed to act parallel to the wind direction.
- Linear interpolation is permitted for h/D values other than shown.
- Notation:
 D : diameter of circular cross-section and least horizontal dimension of square, hexagonal or octagonal cross-sections at elevation under consideration, in meters;
 D' : depth of protruding elements such as ribs and spoilers, m; and
 h : height of structure, m; and
 q_z : velocity pressure evaluated at height z above ground, N/m^2
- For rooftop equipment on buildings with a mean roof height of $h \leq 18$ m, use Section 207D.5.1.

Figure 207D.5-1
Other Structures Force Coefficients, C_f
Chimneys, Tanks, Rooftop Equipment, & Similar Structures

ϵ	Flat-Sided Members	Rounded Members	
		$D\sqrt{q_z} \leq 5.3$	$D\sqrt{q_z} > 5.3$
< 0.1	2.0	1.2	0.8
0.1 to 0.29	1.8	1.3	0.9
0.3 to 0.7	1.6	1.5	1.1

Notes:

- Signs with openings comprising 30% or more of the gross area are classified as open signs.
- The calculation of the design wind forces shall be based on the area of all exposed members and elements projected on a plane normal to the wind direction. Forces shall be assumed to act parallel to the wind direction.
- The area A_f consistent with these force coefficients is the solid area projected normal to the wind direction.
- Notation:
 ϵ : ratio of solid area to gross area;
 D : diameter of a typical round member, m;
 q_z : velocity pressure evaluated at height z above ground, N/m^2

Figure 207D.5-2
Other Structures Force Coefficients, C_f All Heights of Open Signs & Lattice Frameworks

Tower Cross Section	C_f
Square	$4.0\epsilon^2 - 5.9\epsilon + 4.0$
Triangle	$3.4\epsilon^2 - 4.7\epsilon + 3.4$

Notes:

1. For all wind directions considered, the area A_f consistent with the specified force coefficients shall be the solid area of a tower face projected on the plane of that face for the tower segment under consideration.
2. The specified force coefficients are for towers with structural angles or similar flat sided members.
3. For towers containing rounded members, it is acceptable to multiply the specified force coefficients by the following factor when determining wind forces on such members:

$$0.51\epsilon^2 + 0.57, \text{ but not } > 1.0$$

4. Wind forces shall be applied in the directions resulting in maximum member forces and reactions. For towers with square cross-sections, wind forces shall be multiplied by the following factor when the wind is directed along a tower diagonal:

$$1 + 0.75\epsilon, \text{ but not } > 1.2$$

5. Wind forces on tower appurtenances such as ladders, conduits, lights, elevators, etc., shall be calculated using appropriate force coefficients for these elements.
6. Notation:

ϵ : ratio of solid area to gross area of one tower face for the segment under consideration.

Figure 207D.5-2
Other Structures Force Coefficients, C_f Trussed Towers of All Heights

207E Wind Loads – Components and Cladding (C&C)

Commentary:

In developing the set of pressure coefficients applicable for the design of components and cladding (C&C) as given in Figures 207E.4-1, 207E.4-2A, 207E.4-2B, 207E.4-2C, 207E.4-3, 207E.4-4, 207E.4-5A, 207E.4-5B, and 207E.4-6, an envelope approach was followed but using different methods than for the MWFRS of Figure 207C.4-1. Because of the small effective area that may be involved in the design of a particular component (consider, e.g., the effective area associated with the design of a fastener), the point wise pressure fluctuations may be highly correlated over the effective area of interest. Consider the local purlin loads shown in Figure C207C.4-1. The approach involved spatial averaging and time averaging of the point pressures over the effective area transmitting loads to the purlin while the building model was permitted to rotate in the wind tunnel through 360°. As the induced localized pressures may also vary widely as a function of the specific location on the building, height above ground level, exposure, and more importantly, local geometric discontinuities and location of the element relative to the boundaries in the building surfaces (walls, roof lines), these factors were also enveloped in the wind tunnel tests. Thus, for the pressure coefficients given in Figures 207E.4-1, 207E.4-2A, 207E.4-2B, 207E.4-2C, 207E.4-3, 207E.4-4, 207E.4-5A, 207E.4-5B, and 207E.4-6, the directionality of the wind and influence of exposure have been removed and the surfaces of the building “zoned” to reflect an envelope of the peak pressures possible for a given design application.

As indicated in the discussion for Figure 207C.4-1, the wind tunnel experiments checked both Exposure B and C terrains. Basically (GC_p) values associated with Exposure B terrain would be higher than those for Exposure C terrain because of reduced velocity pressure in Exposure B terrain. The (GC_p) values given in Figures 207E.4-1, 207E.4-2A, 207E.4-2B, 207E.4-2C, 207E.4-3, 207E.4-4, 207E.4-5A, 207E.4-5B, and 207E.4-6 are associated with Exposure C terrain as obtained in the wind tunnel. However, they may also be used for any exposure when the correct velocity pressure representing the appropriate exposure is used as discussed below.

The wind tunnel studies conducted by ESDU (1990) determined that when low-rise buildings ($h < 18$ m) are embedded in suburban terrain (Exposure B), the pressures on components and cladding in most cases are lower than those currently used in the standards and codes, although the values show a very large scatter because of high turbulence and many variables. The results seem to

indicate that some reduction in pressures for components and cladding of buildings located in Exposure B is justified. Hence, the code permits the use of the applicable exposure category when using these coefficients.

The pressure coefficients given in Figure 207E.6-1 for buildings with mean height greater than 18 m were developed following a similar approach, but the influence of exposure was not enveloped (Stathopoulos and Dumitrescu-Brulotte 1989). Therefore, exposure categories B, C, or D may be used with the values of (GC_p) in Figure 207E.6-1 as appropriate.

207E.1 Scope

207E.1.1 Building Types

This chapter applies to the determination of wind pressures on components and cladding (C&C) on buildings.

1. Part 1 is applicable to an enclosed or partially enclosed:

- Low-rise building (see definition in Section 207A.2)
- Building with $h \leq 18$ m

The building has a flat roof, gable roof, multispans gable roof, hip roof, monoslope roof, stepped roof, or sawtooth roof and the wind pressures are calculated from a wind pressure equation.

2. Part 2 is a simplified approach and is applicable to an enclosed:

- Low-rise building (see definition in Section 207A.2)
- Building with $h \leq 18$ m

The building has a flat roof, gable roof, or hip roof and the wind pressures are determined directly from a table.

3. Part 3 is applicable to an enclosed or partially enclosed:

- Building with $h > 18$ m

The building has a flat roof, pitched roof, gable roof, hip roof, mansard roof, arched roof, or domed roof and the wind pressures are calculated from a wind pressure equation.

4. Part 4 is a simplified approach and is applicable to an enclosed:

- Building with $h \leq 48 \text{ m}$

The building has a flat roof, gable roof, hip roof, monoslope roof, or mansard roof and the wind pressures are determined directly from a table.

5. Part 5 is applicable to an open building of all heights having a pitched free roof, monoslope free roof, or trough free roof.
6. Part 6 is applicable to building appurtenances such as roof overhangs and parapets and rooftop equipment.

207E.1.2 Conditions

A building whose design wind loads are determined in accordance with this chapter shall comply with all of the following conditions:

1. The building is a regular-shaped building as defined in Section 207A.2.
2. The building does not have response characteristics making it subject to across wind loading, vortex shedding, or instability due to galloping or flutter; or it does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

207E.1.3 Limitations

The provisions of this chapter take into consideration the load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings. The loads on buildings not meeting the requirements of Section 207E.1.2, or having unusual shapes or response characteristics, shall be determined using recognized literature documenting such wind load effects or shall use the wind tunnel procedure specified in Section 207F.

207E.1.4 Shielding

There shall be no reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

207E.1.5 Air-Permeable Cladding

Design wind loads determined from Section 207E shall be used for air-permeable cladding unless approved test data or recognized literature demonstrates lower loads for the type of air-permeable cladding being considered.

Commentary:

Air-permeable roof or wall claddings allow partial air pressure equalization between their exterior and interior surfaces. Examples include siding, pressure-equalized rain screen walls, shingles, tiles, concrete roof pavers, and aggregate roof surfacing.

The peak pressure acting across an air-permeable cladding material is dependent on the characteristics of other components or layers of a building envelope assembly. At any given instant the total net pressure across a building envelope assembly will be equal to the sum of the partial pressures across the individual layers as shown in Figure C207E.1-1. However, the proportion of the total net pressure borne by each layer will vary from instant to instant due to fluctuations in the external and internal pressures and will depend on the porosity and stiffness of each layer, as well as the volumes of the air spaces between the layers. As a result, although there is load sharing among the various layers, the sum of the peak pressures across the individual layers will typically exceed the peak pressure across the entire system. In the absence of detailed information on the division of loads, a simple, conservative approach is to assign the entire differential pressure to each layer designed to carry load.

To maximize pressure equalization (reduction) across any cladding system (irrespective of the permeability of the cladding itself), the layer or layers behind the cladding should be:

- *relatively stiff in comparison to the cladding material and*
- *relatively air-impermeable in comparison to the cladding material.*

Furthermore, the air space between the cladding and the next adjacent building envelope surface behind the cladding (e.g., the exterior sheathing) should be as small as practicable and compartmentalized to avoid communication or venting between different pressure zones of a building's surfaces.

The design wind pressures derived from Section 207E represent the pressure differential between the exterior and interior surfaces of the exterior envelope (wall or roof system). Because of partial air-pressure equalization provided by air-permeable claddings, the components and cladding pressures derived from Section 207E can overestimate the load on air-permeable cladding elements. The designer may elect either to use the loads derived from Section 207E or to use loads derived by an approved alternative method. If the designer desires to determine the

pressure differential across a specific cladding element in combination with other elements comprising a specific building envelope assembly, appropriate full-scale pressure measurements should be made on the applicable building envelope assembly, or reference should be made to recognized literature (Cheung and Melbourne 1986, Haig 1990, Baskaran 1992, Southern Building Code Congress International 1994, Peterka et al. 1997, ASTM 2006, 2007, and Kala et al. 2008) for documentation pertaining to wind loads. Such alternative methods may vary according to a given cladding product or class of cladding products or assemblies because each has unique features that affect pressure equalization.

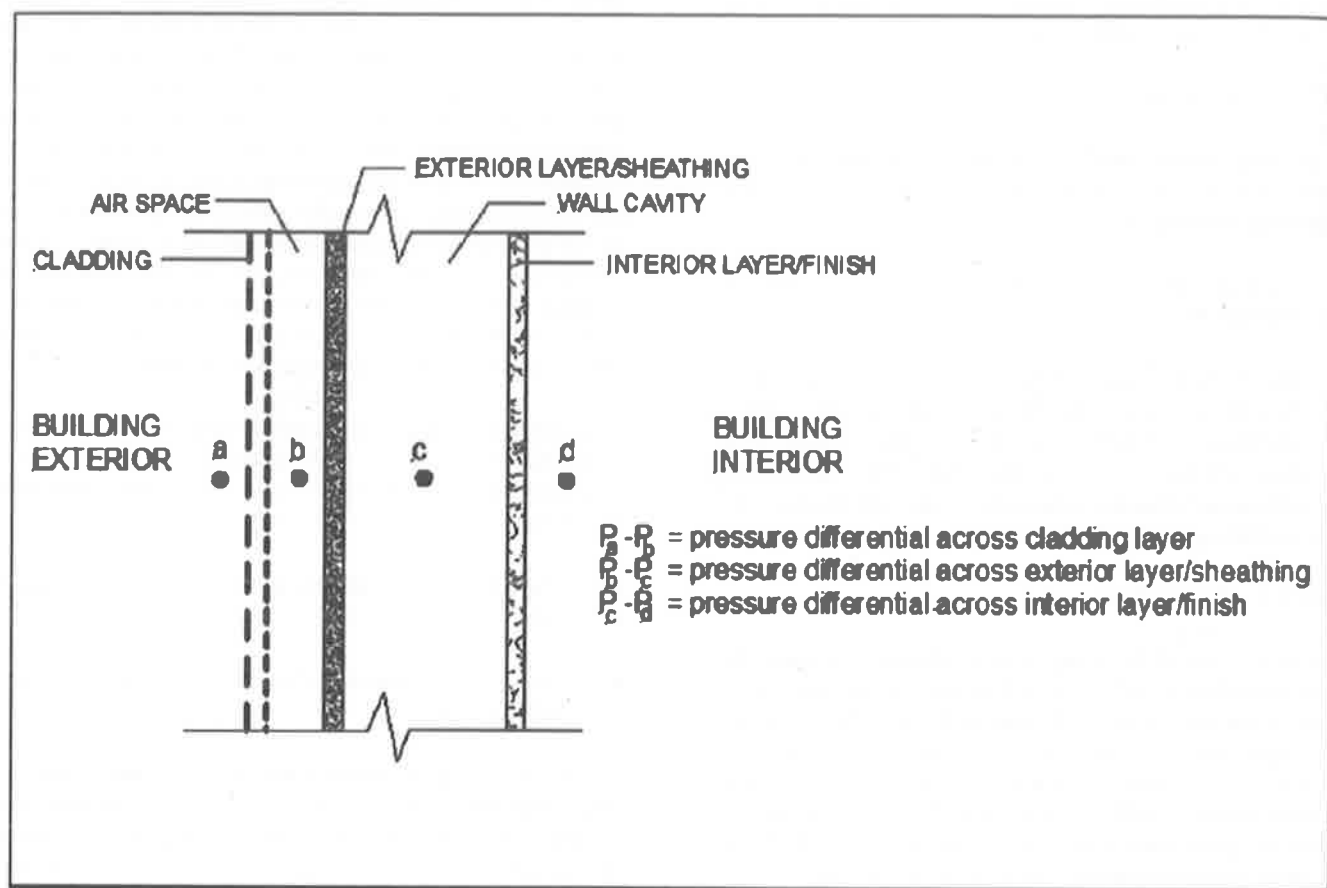


Figure C207E.1-1
Distribution of Net Components and Cladding Pressure Acting on a Building Surface
(Building Envelope) Comprised of Three Components (Layers)

207E.2 General Requirements

207E.2.1 Wind Load Parameters Specified in Section 207A

The following wind load parameters are specified in Section 207A:

- Basic Wind Speed V (Section 207A.5)
- Wind directionality factor K_d (Section 207A.6)
- Exposure category (Section 207A.7)
- Topographic factor K_{zt} (Section 207A.8)
- Gust Effect Factor (Section 207A.9)
- Enclosure classification (Section 207A.10)

- Internal pressure coefficient (GC_{pi}) (Section 207A.11).

207E.2.2 Minimum Design Wind Pressures

The design wind pressure for components and cladding of buildings shall not be less than a net pressure of 0.77 kN/m^2 acting in either direction normal to the surface.

207E.2.3 Tributary Areas Greater than 65 m^2

Component and cladding elements with tributary areas greater than 65 m^2 shall be permitted to be designed using the provisions for MWFRS.

207E.2.4 External Pressure Coefficients

Combined gust effect factor and external pressure coefficients for components and cladding, (GC_p), are given in the figures associated with this chapter. The pressure coefficient values and gust effect factor shall not be separated.

207E.3 Velocity Pressure

207E.3.1 Velocity Pressure Exposure Coefficient

Based on the exposure category determined in Section 207A.7.3, a velocity pressure exposure coefficient K_z or K_h , as applicable, shall be determined from Table 207E.3-1. For a site located in a transition zone between exposure categories, that is, near to a change in ground surface roughness, intermediate values of K_z or K_h , between those shown in Table 207E.3-1, are permitted, provided that they are determined by a rational analysis method defined in the recognized literature.

Commentary:

See commentary, Section C207B.3.1.

207E.3.2 Velocity Pressure

Velocity pressure, q_z , evaluated at height z shall be calculated by the following equation:

$$q_z = 0.613 K_z K_{zt} K_d V^2 \quad (\text{N/m}^2); V \text{ in m/s} \quad (207E.3-1)$$

where

- K_d = wind directionality factor, see Section 207A.6
- K_z = velocity pressure exposure coefficient, see Section 207E.3.1

- K_{zt} = topographic factor defined, see Section 207A.8
- V = basic wind speed, see Section 207A.5
- q_h = velocity pressure calculated using Equation 207E.3-1 at height h

The numerical coefficient 0.613 shall be used except where sufficient climatic data are available to justify the selection of a different value of this factor for a design application.

Commentary:

See commentary, Section C207B.3.2.

Figures 207E.4-1, 207E.4-2A, 207E.4-2B, and 207E.4-2C. The pressure coefficient values provided in these figures are to be used for buildings with a mean roof height of 18 m or less. The values were obtained from wind-tunnel tests conducted at the University of Western Ontario (Davenport et al. 1977, 1978), at the James Cook University of North Queensland (Best and Holmes 1978), and at Concordia University (Stathopoulos 1981, Stathopoulos and Zhu 1988, Stathopoulos and Luchian 1990, 1992, and Stathopoulos and Saathoff 1991). These coefficients were refined to reflect results of full-scale tests conducted by the National Bureau of Standards (Marshall 1977) and the Building Research Station, England (Eaton and Mayne 1975). Pressure coefficients for hemispherical domes on the ground or on cylindrical structures were based on wind-tunnel tests (Taylor 1991). Some of the characteristics of the values in the figure are as follows:

1. The values are combined values of (GC_p). The gust effect factors from these values should not be separated.
2. The velocity pressure q_h evaluated at mean roof height should be used with all values of (GC_p).
3. The values provided in the figure represent the upper bounds of the most severe values for any wind direction. The reduced probability that the design wind speed may not occur in the particular direction for which the worst pressure coefficient is recorded has not been included in the values shown in the figure.
4. The wind-tunnel values, as measured, were based on the mean hourly wind speed. The values provided in the figures are the measured values divided by $(1.53)^2$ (see Figure C207A.5-1) to adjust for the reduced pressure coefficient values associated with a 3-s gust speed.

Each component and cladding element should be designed for the maximum positive and negative pressures (including applicable internal pressures) acting on it. The

pressure coefficient values should be determined for each component and cladding element on the basis of its location on the building and the effective area for the element. Research (Stathopoulos and Zhu 1988, 1990) indicated that the pressure coefficients provided generally apply to facades with architectural features, such as balconies, ribs, and various facade textures. In ASCE 7-02, the roof slope range and values of (GC_p) were updated based on subsequent studies (Stathopoulos et al. 1999, 2000, 2001).

Figures 207E.4-4, 207E.4-5A, and 207E.4-5B. These figures present values of (GC_p) for the design of roof components and cladding for buildings with multispans gable roofs and buildings with monoslope roofs. The coefficients are based on wind tunnel studies (Stathopoulos and Mohammadian 1986, Surry and Stathopoulos 1988, and Stathopoulos and Saathoff 1991).

Figure 207E.4-6. The values of (GC_p) in this figure are for the design of roof components and cladding for buildings with sawtooth roofs and mean roof height, h , less than or equal to 18 m. Note that the coefficients for corner zones on segment A differ from those coefficients for corner zones on the segments designated as B, C, and D. Also, when the roof angle is less than or equal to 10° , values of (GC_p) for regular gable roofs (Figure 207E.4-2A) are to be used. The coefficients included in Figure 207E.4-6 are based on wind tunnel studies reported by Saathoff and Stathopoulos (1992).

Figure 207E.4-7. This figure for cladding pressures on dome roofs is based on Taylor (1991). Negative pressures are to be applied to the entire surface, because they apply along the full arc that is perpendicular to the wind direction and that passes through the top of the dome. Users are cautioned that only three shapes were available to define values in this figure ($h_D/D = 0.5$, $f/D = 0.5$; $h_D/D = 0.0$, $f/D = 0.5$; and $h_D/D = 0.0$, $f/D = 0.33$).

Figure 207E.6-1. The pressure coefficients shown in this figure reflect the results obtained from comprehensive wind tunnel studies carried out (Stathopoulos and Dumitrescu-Brulotte 1989). The availability of more comprehensive wind tunnel data has also allowed a simplification of the zoning for pressure coefficients, flat roofs are now divided into three zones, and walls are represented by two zones.

The external pressure coefficients and zones given in Figure 207E.6-1 were established by wind tunnel tests on isolated “box-like” buildings (Akins and Cermak 1975 and Peterka and Cermak 1975). Boundary-layer wind-tunnel tests on high-rise buildings (mostly in downtown city centers) show that variations in pressure coefficients and the distribution of pressure on the different building facades are obtained (Templin and Cermak 1978). These variations are due to building geometry, low attached buildings, nonrectangular cross-sections, setbacks, and sloping surfaces. In addition, surrounding buildings contribute to the variations in pressure. Wind tunnel tests indicate that pressure coefficients are not distributed symmetrically and can give rise to torsional wind loading on the building.

Boundary-layer wind-tunnel tests that include modeling of surrounding buildings permit the establishment of more exact magnitudes and distributions of (GC_p) for buildings that are not isolated or “boxlike” in shape.

Table 207E.3-1
Velocity Pressure Exposure Coefficients, K_h and K_z

Height above ground level, z (m)	Exposure		
	B	C	D
0 - 4.5	0.70	0.85	1.03
6.0	0.70	0.90	1.08
7.5	0.70	0.94	1.12
9.0	0.70	0.98	1.16
12.0	0.76	1.04	1.22
15.0	0.81	1.09	1.27
18.0	0.85	1.13	1.31
21.0	0.89	1.17	1.34
24.0	0.93	1.21	1.38
27.0	0.96	1.24	1.40
30.0	0.99	1.26	1.43
36.0	1.04	1.31	1.48
42.0	1.09	1.36	1.52
48.0	1.13	1.39	1.55
54.0	1.17	1.43	1.58
60.0	1.20	1.46	1.61
75.0	1.28	1.53	1.68
90.0	1.35	1.59	1.73
105.0	1.41	1.64	1.78
120.0	1.47	1.69	1.82
135.0	1.52	1.73	1.86
150.0	1.56	1.77	1.89

Notes:

1. The velocity pressure exposure coefficient K_z may be determined from the following formula:

For $4.57 \text{ m} \leq z \leq z_g$

For $z < 4.57 \text{ m}$

$$K_z = 2.01(z/z_g)^{2/\alpha}$$

$$K_z = 2.01(4.57/z_g)^{2/\alpha}$$

2. α and z_g are tabulated in Table 207A.9-1.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposure categories are defined in Section 207A.7.

Part 1: Low-Rise Buildings*Commentary:*

The component and cladding tables in Figure 207E.5-1 are a tabulation of the pressures on an enclosed, regular, 9-m high building with a roof as described. The pressures can be modified to a different exposure and height with the same adjustment factors as the MWFRS pressures. For the designer to use this method for the design of the components and cladding, the building must conform to all five requirements in Section 207E.6; otherwise one of the other procedures specified in Section 207E.1.1 must be used.

207E.4 Building Types

The provisions of Section 207E.4 are applicable to an enclosed and partially enclosed:

- Low-rise building (see definition in Section 207A.2)
- Building with $h \leq 18$ m

The building has a flat roof, gable roof, multispans gable roof, hip roof, monoslope roof, stepped roof, or sawtooth roof. The steps required for the determination of wind loads on components and cladding for these building types are shown in Table 207E.4-1.

207E.4.1 Conditions

For the determination of the design wind pressures on the components and claddings using the provisions of Section 207E.4.2 the conditions indicated on the selected figure(s) shall be applicable to the building under consideration.

207E.4.2 Design Wind Pressures

Design wind pressures on component and cladding elements of low-rise buildings and buildings with $h \leq 18$ m shall be determined from the following equation:

$$p = q_z [(GC_p) - (GC_{pi})] \text{ (N/m}^2\text{)} \quad 207E.4-1$$

where

q_z = velocity pressure evaluated at mean roof height h as defined in Section 207E.3

(GC_p) = external pressure coefficients given in:

- Figure 207E.4-1 (walls)
- Figures. 207E.4-2A to 207E.4-2C (flat roofs, gable roofs, and hip roofs)
- Figure 207E.4-3 (stepped roofs)
- Figure 207E.4-4 (multispans gable roofs)
- Figures 207E.4-5A and 207E.4-5B (monoslope roofs)
- Figure 207E.4-6 (sawtooth roofs)
- Figure 207E.4-7 (domed roofs)
- Figure 207B.4-3, footnote 4 (arched roofs)

(GC_{pi}) = internal pressure coefficient given in Table 207A.11-1

User Note:

Use Part 1 of Section 207E to determine wind pressures on C&C of enclosed and partially enclosed low-rise buildings having roof shapes as specified in the applicable figures. The provisions in Part 1 are based on the Envelope Procedure with wind pressures calculated using the specified equation as applicable to each building surface. For buildings for which these provisions are applicable this method generally yields the lowest wind pressures of all analytical methods contained in this code.

Table 207E.4-1
Steps to Determine C&C Wind Loads
Enclosed and Partially Enclosed
Low-rise Buildings

Step 1:	Determine risk category of building or other structure, see Table 103-1
Step 2:	Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C
Step 3:	Determine wind load parameters: ➤ Wind directionality factor, K_d, see Section 207A.6 and Table 207A.6-1 ➤ Exposure category B, C or D, see Section 207A.7 ➤ Topographic factor, K_{zt}, see Section 207A.8 and Figure 207A.8-1 ➤ Enclosure classification, see Section 207A.10 ➤ Internal pressure coefficient, (GC_{pi}), see Section 207A.11 and Table 207A.11-1
Step 4:	Determine velocity pressure exposure coefficient K_z or K_h , see Table 207E.3-1
Step 5:	Determine velocity pressure, q_h , see Equation 207E.3-1
Step 6:	Determine external pressure coefficient, (GC_p) ➤ Walls, see Figure 207E.4-1 ➤ Flat roofs, gable roofs, hip roofs, see Figure 207E.4-2 ➤ Stepped roofs, see Figure 207E.4-3 ➤ Multispan gable roofs, see Figure 207E.4-4 ➤ Monoslope roofs, see Figure 207E.4-5 ➤ Sawtooth roofs, see Figure 207E.4-6 ➤ Domed roofs, see Figure 207E.4-7 ➤ Arched roofs, see Figure 207B.4-3 footnote 4
Step 7:	Calculate wind pressure, p , Equation 207E.4-1

Part 2: Low-Rise Buildings (Simplified)

207E.5 Building Types

The provisions of Section 207E.5 are applicable to an enclosed:

- Low-rise building (see definition in Section 207A.2)
- Building with $h \leq 18$ m

The building has a flat roof, gable roof, or hip roof. The steps required for the determination of wind loads on components and cladding for these building types are shown in Table 207E.5-1.

207E.5.1 Conditions

For the design of components and cladding the building shall comply with all the following conditions:

1. The mean roof height h must be less than or equal to 18 m (i.e. $h \leq 18$ m).
2. The building is enclosed as defined in Section 207A.2 and conforms to the wind-borne debris provisions of Section 207A.10.3.
3. The building is a regular-shaped building or structure as defined in Section 207A.2.
4. The building does not have response characteristics making it subject to across wind loading, vortex shedding, or instability due to galloping or flutter; and it does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
5. The building has either a flat roof, a gable roof with $\theta \leq 45^\circ$, or a hip roof with $\theta \leq 27^\circ$.

207E.5.2 Design Wind Pressures

Net design wind pressures, p_{net} , for component and cladding of buildings designed using the procedure specified herein represent the net pressures (sum of internal and external) that shall be applied normal to each building surface as shown in Figure 207E.5-1. p_{net} shall be determined by the following equation:

$$p_{net} = \lambda K_{zt} p_{net9} \quad (207E.5-1)$$

where

- λ = adjustment factor for building height and exposure from Figure 207E.5-1
- K_{zt} = topographic factor as defined in Section 207A.8 evaluated at 0.33 mean roof height, **0.33h**
- p_{net30} = net design wind pressure for Exposure B, at $h = 9$ m, from Figure 207E.5-1

User Note:

Part 2 of Section 207E is a simplified method to determine wind pressures on C&C of enclosed low-rise buildings having flat, gable or hip roof shapes. The provisions of Part 2 are based on the Envelope Procedure of Part 1 with wind pressures determined from a table and adjusted as appropriate.

Table 207E.5-1
Steps to Determine C&C
Wind Loads Enclosed Low-rise Buildings
(Simplified Method)

Step 1:	Determine risk category, see Table 103-1
Step 2:	Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C
Step 3:	Determine wind load parameters: ➤ Exposure category B, C or D, see Section 207A.7 ➤ Topographic factor, K_{zt}, see Section 207A.8 and Figure 207A.8-1
Step 4:	Enter figure to determine wind pressures at $h = 9$ m., p_{net9} , see Figure 207E.5-1
Step 5:	Enter figure to determine adjustment for building height and exposure, λ , see Figure 207E.5-1
Step 6:	Determine adjusted wind pressures, p_{net} , see Equation 207E.5-1.

Part 3: Buildings with $h > 18$ m*Commentary:*

In Equation 207E.6-1 a velocity pressure term, q_i , appears that is defined as the "velocity pressure for internal pressure determination." The positive internal pressure is dictated by the positive exterior pressure on the windward face at the point where there is an opening. The positive exterior pressure at the opening is governed by the value of q at the level of the opening, not q_h . For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$). For low buildings this does not make much difference, but for the example of a 91.5-m-tall building in Exposure B with the highest opening at 18 m, the difference between $q_{9.15}$ and q_{18} represents a 59 percent increase in internal pressure. This is unrealistic and represents an unnecessary degree of conservatism. Accordingly, $q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact protective system, q_i should be treated as an opening.

207E.6 Building Types

The provisions of Section 207E.6 are applicable to an enclosed or partially enclosed building with a mean roof height $h > 18$ m with a flat roof, pitched roof, gable roof, hip roof, mansard roof, arched roof, or domed roof. The steps required for the determination of wind loads on components and cladding for these building types are shown in Table 207E.6-1.

207E.6.1 Conditions

For the determination of the design wind pressures on the component and cladding using the provisions of Section 207E.6.2, the conditions indicated on the selected figure(s) shall be applicable to the building under consideration.

207E.6.2 Design Wind Pressures

Design wind pressures on component and cladding for all buildings with $h > 18$ m shall be determined from the following equation:

$$p = q(GC_p) - (GC_{pi}) \text{ (N/m}^2\text{)} \quad 207E.6-1$$

where

- q = q_z for windward walls calculated at height z above the ground
- q = q_h for leeward walls, side walls, and roofs evaluated at height h
- q_i = q_h for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings

- q_i = q_z for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$)

(GC_p) = External pressure coefficients given in:

- Figure 207E.6-1 for walls and flat roofs
- Figure 207B.4-3, footnote 4, for arched roofs
- Figure 207E.4-7 for domed roofs
- Note 6 of Figure 207E.6-1

(GC_{pi}) = Internal pressure coefficient given in Table 207A.11-1

q and q_i shall be evaluated using exposure defined in Section 207A.11-1

Exception:

In buildings with a mean roof height h greater than 18 m and less than 27.4 m, (GC_p) values from Figures 207E.4-1 through 207E.4-6 shall be permitted to be used if the height to width ratio is one or less.

User Note:

Section Part 3 of Section 207E for determining wind pressures for C&C of enclosed and partially enclosed buildings with $h > 18$ m having roof shapes as specified in the applicable figures. These provisions are based on the Directional Procedure with wind pressures calculated from the specified equation applicable to each building surface.

Table 207E.6-1
Steps to Determine C&C Wind Loads
Enclosed and Partially Enclosed
Building with $h > 18$ m

- | | |
|---------|--|
| Step 1: | Determine risk category, see Table 103-1 |
| Step 2: | Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C |
| Step 3: | Determine wind load parameters: <ul style="list-style-type: none"> ➤ Wind directionality factor, K_d, see Section 207A.6 and Table 207A.6-1 ➤ Exposure category B, C or D, see Section 207A.7 ➤ Topographic factor, K_{zt}, see Section 207A.8 and Figure 207A.8-1 ➤ Enclosure classification, see Section 207A.10 ➤ Internal pressure coefficient, (GC_{pi}), see Section 207A.11 and Table 207A.11-1 |
| Step 4: | Determine velocity pressure exposure coefficient K_z or K_h , see Table 207E.3-1 |
| Step 5: | Determine velocity pressure, q_h , see Table 207E.3-1 |
| Step 6: | Determine external pressure coefficient, (GC_p) <ul style="list-style-type: none"> ➤ Walls and flat roofs, $(\theta < 10^\circ)$, see Figure 207E.6-1 ➤ Gable and hip roofs, see Figure 207E.4-2 per Note 6 of Figure 207E.6-1 ➤ Arched roofs, see Figure 207B.4-3, footnote 4 ➤ Domed roofs, see Figure 207E.4-7 |
| Step 7: | Calculate wind pressure, p , Equation 207E.6-1 |

Part 4: Buildings with $h \leq 48$ m (Simplified)

Commentary:

This section has been added to ASCE 7-10 to cover the common practical case of enclosed buildings up to height $h = 49$ m. Table 207E.7-2 includes wall and roof pressures for flat roofs ($\theta < 10^\circ$), gable roofs, hip roofs, monoslope roofs, and mansard roofs. Pressures are derived from Figure 207E.6-1 (flat roofs), Figure 207E.4-2A, B, and C (gable and hip roofs), and Figure 207E.4-5A and B (monoslope roofs) of Part 3. Pressures were selected for each zone that encompasses the largest pressure coefficients for the comparable zones from the different roof shapes. Thus, for some cases, the pressures tabulated are conservative in order to maintain simplicity. The (GC_p) values from these figures were combined with an internal pressure coefficient (+ or - 0.18) to obtain a net coefficient from which pressures were calculated. The tabulated pressures are applicable to the entire zone shown in the various figures.

Pressures are shown for an effective wind area of 0.93 m^2 . A reduction factor is also shown to obtain pressures for larger effective wind areas. The reduction factors are based on the graph of external pressure coefficients shown in the figures in Part 3 and are based on the most conservative reduction for each zone from the various figures.

207E.7 Building Types

The provisions of Section 207E.7 are applicable to an enclosed building having a mean roof height $h \leq 49$ m with a flat roof, gable roof, hip roof, monoslope roof, or mansard roof. The steps required for the determination of wind loads on components and cladding for these building types are shown in Table 207E.7-1.

207E.7.1 Wind Loads—Components and Cladding

207E.7.1.1 Wall and Roof Surfaces

Design wind pressures on the designated zones of walls and roofs surfaces shall be determined from Table 207E.7-2 based on the applicable basic wind speed V , mean roof height h , and roof slope θ . Tabulated pressures shall be multiplied by the exposure adjustment factor (**EAF**) shown in the table if exposure is different than Exposure C. Pressures in Table 207E.7-2 are based on an effective wind area of 0.93 m^2 . Reductions in wind pressure for larger effective wind areas may be taken based on the reduction multipliers (**RF**) shown in the table. Pressures are to be applied over the entire zone shown in the figures. Final design wind pressure shall be determined from the following equation:

$$p = p_{\text{table}}(\text{EAF})(\text{RF})K_{zt} \quad (207\text{E}.7-1)$$

where

- RF** = effective area reduction factor from Table 207E.7-2
EAF = Exposure adjustment factor from Table 207E.7-2
K_{zt} = topographic factor as defined in Section 207A.8

207E.7.1.2 Parapets

Design wind pressures on parapet surfaces shall be based on wind pressures for the applicable edge and corner zones in which the parapet is located, as shown in Table 207E.7-2, modified based on the following two load cases:

- Load Case A shall consist of applying the applicable positive wall pressure from the table to the front surface of the parapet while applying the applicable negative edge or corner zone roof pressure from the table to the back surface.
- Load Case B shall consist of applying the applicable positive wall pressure from the table to the back of the parapet surface and applying the applicable negative wall pressure from the table to the front surface.

User Note:

Part 4 of Section 207E is a simplified method for determining wind pressures for C&C of enclosed and partially enclosed buildings with $h \leq 49 \text{ m}$, having roof shapes as specified in the applicable figures. These provisions are based on the Directional Procedure from Part 3 with wind pressures selected directly from a table and adjusted as applicable.

Table 207E.7-1
Steps to Determine C&C Wind Loads
Enclosed Building with $h > 48.8 \text{ m}$

Step 1:	Determine risk category, see Table 103-1
Step 2:	Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C
Step 3:	Determine wind load parameters: ➤ Exposure category B, C or D, see Section 207A.7
Step 4:	Enter Table 207E.7-2 to determine pressure on walls and roof, p , using Equation 207E.7-1. Roof types are: ➤ Flat roof ($\theta < 10^\circ$) ➤ Gable roof ➤ Hip roof ➤ Monoslope roof ➤ Mansard roof
Step 5:	Determine topographic factors, K_{zt} , and apply factor to pressures determined from tables (if applicable), see Section 207A.8

Pressures in Table 207E.7-2 are based on an effective wind area of 0.93 m^2 . Reduction in wind pressure for larger effective wind area may be taken based on the reduction factor shown in the table. Pressures are to be applied to the parapet in accordance with Figure 207E.7-1. The height h to be used with Figure 207E.7-1 to determine the pressures shall be the height to the top of the parapet. Determine final pressure from Equation 207E.7-1.

Commentary:

Parapet component and cladding wind pressures can be obtained from the tables as shown in the parapet figures

from the table. The pressures obtained are slightly conservative based on the net pressure coefficients for parapets compared to roof zones from Part 3. Two load cases must be considered based on pressures applied to both windward and leeward parapet surfaces as shown in Figure 207E.7-1.

207E.7.1.3 Roof Overhangs

Design wind pressures on roof overhangs shall be based on wind pressures shown for the applicable zones in Table 207E.7-2 modified as described herein. For Zones 1 and 2, a multiplier of 1.0 shall be used on pressures shown in Table 207E.7-2. For Zone 3, a multiplier of 1.15 shall be used on pressures shown in Table 207E.7-2.

Pressures in Table 207E.7-2 are based on an effective wind area of 0.93 m^2 . Reductions in wind pressure for larger effective wind areas may be taken based on the reduction multiplier shown in Table 207E.7-2. Pressures on roof overhangs include the pressure from the top and bottom surface of overhang. Pressures on the underside of the overhangs are equal to the adjacent wall pressures. Refer to the overhang drawing shown in Figure 207E.7-2. Determine final pressure from Equation 207E.7-1.

Commentary:

Component and cladding pressures for roof overhangs can be obtained from the tables as shown in Figure 207E.7-2. These pressures are slightly conservative and are based on the external pressure coefficients contained in Figure 207E.4-2A to 207E.4-2C from Part 3.

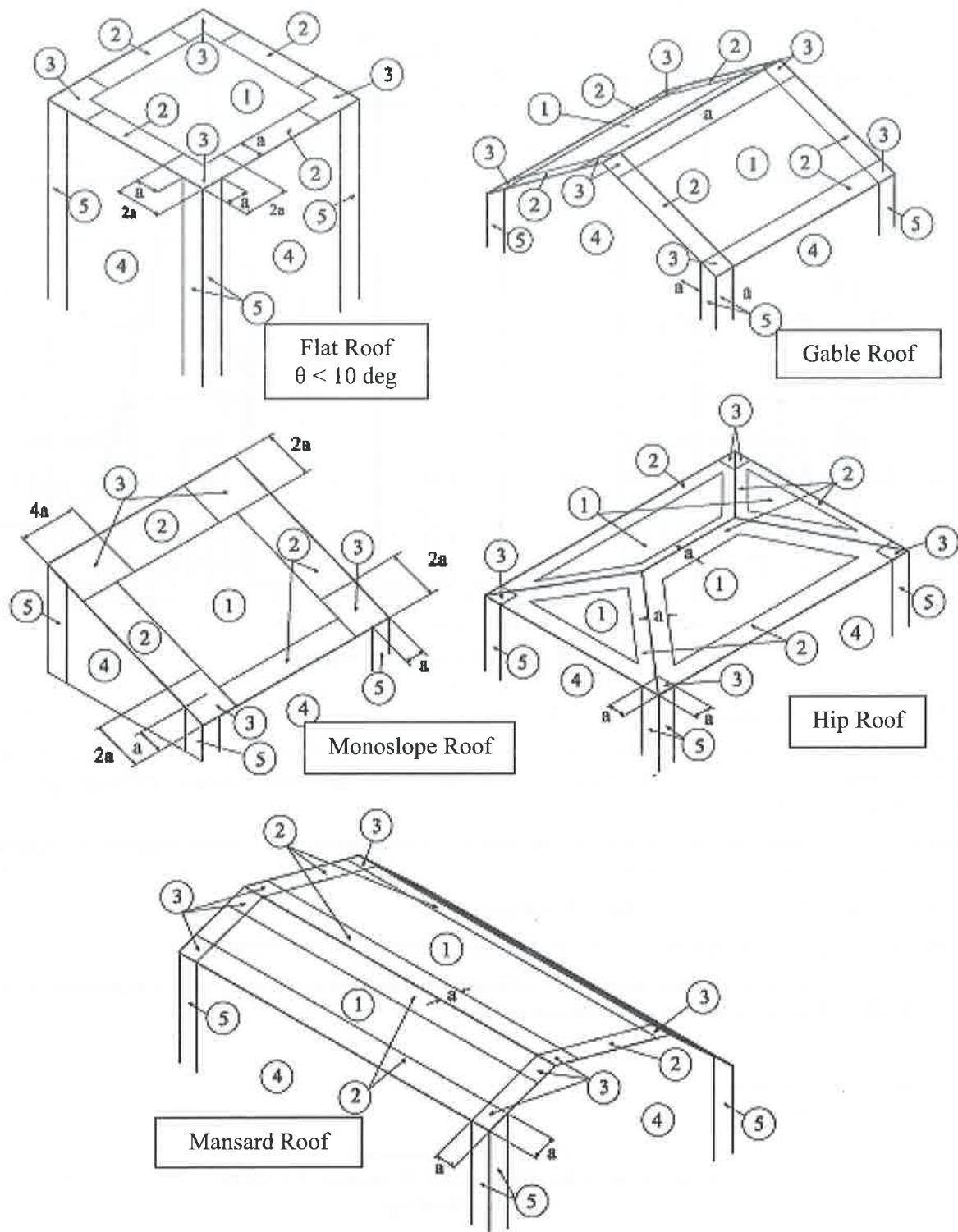
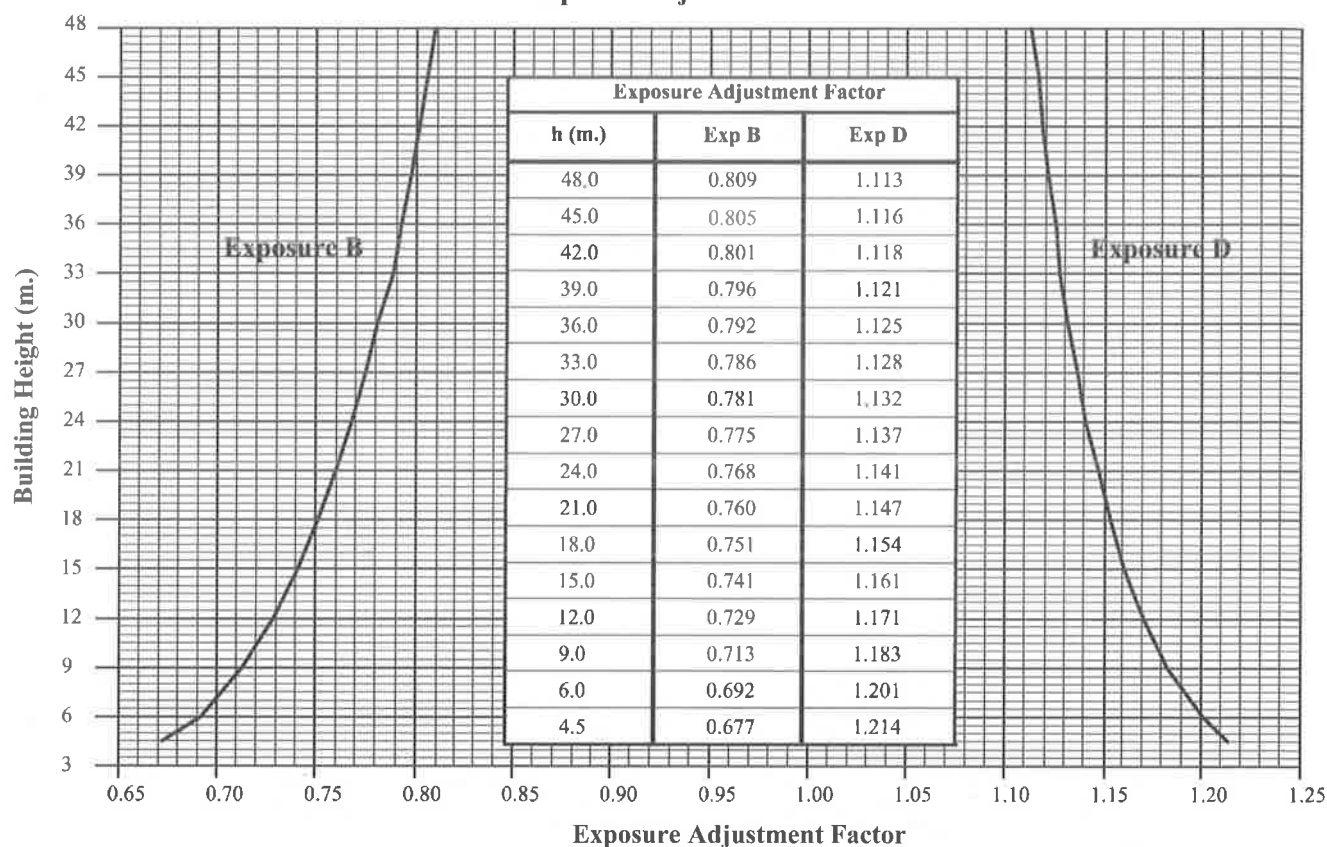


Figure 207E.7-2
C & C Zones C&C Wall and Roof Pressures, $h \leq 48$ m
Enclosed Buildings

Roof and Wall Pressures-Components and Cladding Exposure Adjustment Factor



Notes to Component and Cladding Wind Pressure Table:

- For each roof form, Exposure C, V and h determine roof and wall cladding pressures for the applicable zone from tables above. For other exposures B or D, multiply pressures from table by the appropriate exposure adjustment factor determined from figure above.
- Interpolation between h values is permitted. For pressures at other V values than shown in the table, multiply table value for any given V' in the table as shown above:

$$\text{Pressure at desired } V = \text{pressure from table at } V \times [V_{\text{desired}}/V]^2$$

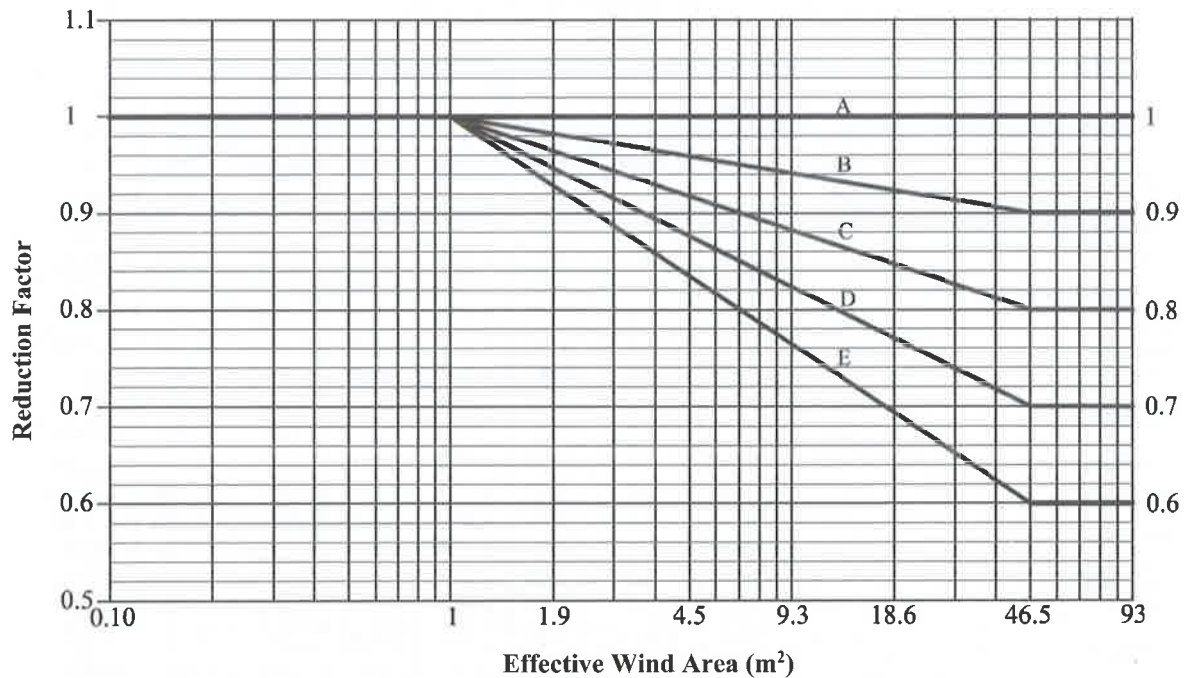
- Where two load cases are shown, both positive and negative pressures shall be considered.
- Pressures are shown for an effective wind area = 0.93 m^2 . For larger effective wind areas, the pressure shown may be reduced by the reduction coefficient applicable to each zone.

Notation:

h mean roof height (m)
 V Basic wind speed (kph)

Table 207E.7-2 (continued)
 C & C Zones C&C Wall and Roof Pressures, $h \leq 48 \text{ m}$
 Enclosed Buildings

**Reduction Factors
Effective Wind Area**



**Reduction Factors
Effective Wind Area**

Roof Form	Sign Pressure	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
Flat	Minus	D	D	D	C	E
Flat	Plus	NA	NA	NA	D	D
Gable, Mansard	Minus	B	C	C	C	E
Gable, Mansard	Plus	B	B	B	D	D
Hip	Minus	B	C	C	C	E
Hip	Plus	B	B	B	D	D
Monoslope	Minus	A	B	D	C	E
Monoslope	Plus	C	C	C	D	D
Overhangs	All	A	A	B	NA	NA

Table 207E.7-2 (continued)
C & C Effective Wind Area C&C Wall and Roof Pressures, $h \leq 48$ m
Enclosed Buildings

Table 207E.7-2
 Components and Cladding – Part 4
 C&C, $V = 150$ -200 kph, $h = 4.5$ - 15 m
 Exposure C

V (kph)			150					200				
h (m)	Roof Form	Load Case	Zone					Zone				
			1	2	3	4	5	1	2	3	4	5
15	Flat Roof	1	-1.5579	-2.4453	-3.3328	-1.0649	-1.9523	-2.7696	-4.3473	-5.9249	-1.8932	-3.4708
		2	NA	NA	NA	1.0649	1.0649	NA	NA	NA	1.8932	1.8932
	Gable Roof Mansard Roof	1	-1.1635	-1.9523	-2.9384	-1.2621	-1.9523	-2.0685	-3.4708	-5.2238	-2.2438	-3.4708
		2	0.6705	0.6705	0.6705	1.1635	1.0649	1.1920	1.1920	1.1920	2.0685	1.8932
	Hip Roof	1	-1.0649	-1.8537	-2.7412	-1.2621	-1.9523	-1.8932	-3.2955	-4.8732	-2.2438	-3.4708
		2	0.6705	0.6705	0.6705	1.1635	1.0649	1.1920	1.1920	1.1920	2.0685	1.8932
	Monoslope Roof	1	-1.3607	-1.7551	-3.0370	-1.2621	-1.9523	-2.4191	-3.1202	-5.3990	-2.2438	-3.4708
		2	0.5719	0.5719	0.5719	1.1635	1.0649	1.0167	1.0167	1.0167	2.0685	1.8932
12	Flat Roof	1	-1.4865	-2.3332	-3.1799	-1.0161	-1.8628	-2.6426	-4.1479	-5.6531	-1.8063	-3.3116
		2	NA	NA	NA	1.0161	1.0161	NA	NA	NA	1.8063	1.8063
	Gable Roof Mansard Roof	1	-1.1101	-1.8628	-2.8036	-1.2042	-1.8628	-1.9736	-3.3116	-4.9841	-2.1408	-3.3116
		2	0.6397	0.6397	0.6397	1.1101	1.0161	1.1373	1.1373	1.1373	1.9736	1.8063
	Hip Roof	1	-1.0161	-1.7687	-2.6154	-1.2042	-1.8628	-1.8063	-3.1444	-4.6496	-2.1408	-3.3116
		2	0.6397	0.6397	0.6397	1.1101	1.0161	1.1373	1.1373	1.1373	1.9736	1.8063
	Monoslope Roof	1	-1.2983	-1.6746	-2.8977	-1.2042	-1.8628	-2.3081	-2.9771	-5.1514	-2.1408	-3.3116
		2	0.5457	0.5457	0.5457	1.1101	1.0161	0.9701	0.9701	0.9701	1.9736	1.8063
9	Flat Roof	1	-1.4007	-2.1986	-2.9964	-0.9574	-1.7553	-2.4901	-3.9086	-5.3270	-1.7021	-3.1206
		2	NA	NA	NA	0.9574	0.9574	NA	NA	NA	1.7021	1.7021
	Gable Roof Mansard Roof	1	-1.0461	-1.7553	-2.6418	-1.1347	-1.7553	-1.8597	-3.1206	-4.6966	-2.0173	-3.1206
		2	0.6028	0.6028	0.6028	1.0461	0.9574	1.0717	1.0717	1.0717	1.8597	1.7021
	Hip Roof	1	-0.9574	-1.6667	-2.4645	-1.1347	-1.7553	-1.7021	-2.9629	-4.3814	-2.0173	-3.1206
		2	0.6028	0.6028	0.6028	1.0461	0.9574	1.0717	1.0717	1.0717	1.8597	1.7021
	Monoslope Roof	1	-1.2234	-1.5780	-2.7305	-1.1347	-1.7553	-2.1749	-2.8053	-4.8542	-2.0173	-3.1206
		2	0.5142	0.5142	0.5142	1.0461	0.9574	0.9141	0.9141	0.9141	1.8597	1.7021
7.5	Flat Roof	1	-1.3435	-2.1088	-2.8741	-0.9184	-1.6837	-2.3885	-3.7490	-5.1096	-1.6326	-2.9932
		2	NA	NA	NA	0.9184	0.9184	NA	NA	NA	1.6326	1.6326
	Gable Roof Mansard Roof	1	-1.0034	-1.6837	-2.5340	-1.0884	-1.6837	-1.7838	-2.9932	-4.5049	-1.9350	-2.9932
		2	0.5782	0.5782	0.5782	1.0034	0.9184	1.0280	1.0280	1.0280	1.7838	1.6326
	Hip Roof	1	-0.9184	-1.5986	-2.3639	-1.0884	-1.6837	-1.6326	-2.8420	-4.2025	-1.9350	-2.9932
		2	0.5782	0.5782	0.5782	1.0034	0.9184	1.0280	1.0280	1.0280	1.7838	1.6326
	Monoslope Roof	1	-1.1735	-1.5136	-2.6190	-1.0884	-1.6837	-2.0862	-2.6908	-4.6561	-1.9350	-2.9932
		2	0.4932	0.4932	0.4932	1.0034	0.9184	0.8768	0.8768	0.8768	1.7838	1.6326
6	Flat Roof	1	-1.2864	-2.0191	-2.7518	-0.8793	-1.6120	-2.2869	-3.5895	-4.8921	-1.5632	-2.8658
		2	NA	NA	NA	0.8793	0.8793	NA	NA	NA	1.5632	1.5632
	Gable Roof Mansard Roof	1	-0.9607	-1.6120	-2.4262	-1.0421	-1.6120	-1.7079	-2.8658	-4.3132	-1.8526	-2.8658
		2	0.5536	0.5536	0.5536	0.9607	0.8793	0.9842	0.9842	0.9842	1.7079	1.5632
	Hip Roof	1	-0.8793	-1.5306	-2.2633	-1.0421	-1.6120	-1.5632	-2.7211	-4.0237	-1.8526	-2.8658
		2	0.5536	0.5536	0.5536	0.9607	0.8793	0.9842	0.9842	0.9842	1.7079	1.5632
	Monoslope Roof	1	-1.1235	-1.4492	-2.5076	-1.0421	-1.6120	-1.9974	-2.5763	-4.4579	-1.8526	-2.8658
		2	0.4722	0.4722	0.4722	0.9607	0.8793	0.8395	0.8395	0.8395	1.7079	1.5632
4.5	Flat Roof	1	-1.2149	-1.9069	-2.5990	-0.8304	-1.5225	-2.1598	-3.3901	-4.6204	-1.4763	-2.7066
		2	NA	NA	NA	0.8304	0.8304	NA	NA	NA	1.4763	1.4763
	Gable Roof Mansard Roof	1	-0.9073	-1.5225	-2.2914	-0.9842	-1.5225	-1.6130	-2.7066	-4.0736	-1.7497	-2.7066
		2	0.5229	0.5229	0.5229	0.9073	0.8304	0.9295	0.9295	0.9295	1.6130	1.4763
	Hip Roof	1	-0.8304	-1.4456	-2.1376	-0.9842	-1.5225	-1.4763	-2.5699	-3.8002	-1.7497	-2.7066
		2	0.5229	0.5229	0.5229	0.9073	0.8304	0.9295	0.9295	0.9295	1.6130	1.4763
	Monoslope Roof	1	-1.0611	-1.3687	-2.3683	-0.9842	-1.5225	-1.8864	-2.4332	-4.2103	-1.7497	-2.7066
		2	0.4460	0.4460	0.4460	0.9073	0.8304	0.7928	0.7928	0.7928	1.6130	1.4763

Table 207E.7-2
Components and Cladding – Part 4
C&C, $V = 250$ -300 kph, $h = 4.5$ - 15 m
Exposure C

V (kph)			250					300				
h (m)	Roof Form	Load Case	Zone					Zone				
			1	2	3	4	5	1	2	3	4	5
15	Flat Roof	1	-4.3276	-6.7926	-9.2577	-2.9581	-5.4232	-6.2317	-9.7814	-13.3311	-4.2596	-7.8093
		2	NA	NA	NA	2.9581	2.9581	NA	NA	NA	4.2596	4.2596
	Gable Roof Mansard Roof	1	-3.2320	-5.4232	-8.1621	-3.5059	-5.4232	-4.6541	-7.8093	-11.7534	-5.0485	-7.8093
		2	1.8625	1.8625	1.8625	3.2320	2.9581	2.6820	2.6820	2.6820	4.6541	4.2596
	Hip Roof	1	-2.9581	-5.1493	-7.6143	-3.5059	-5.4232	-4.2596	-7.4149	-10.9646	-5.0485	-7.8093
		2	1.8625	1.8625	1.8625	3.2320	2.9581	2.6820	2.6820	2.6820	4.6541	4.2596
	Monoslope Roof	1	-3.7798	-4.8754	-8.4360	-3.5059	-5.4232	-5.4429	-7.0205	-12.1479	-5.0485	-7.8093
		2	1.5886	1.5886	1.5886	3.2320	2.9581	2.2876	2.2876	2.2876	4.6541	4.2596
12	Flat Roof	1	-4.1291	-6.4810	-8.8330	-2.8224	-5.1744	-5.9458	-9.3327	-12.7196	-4.0642	-7.4511
		2	NA	NA	NA	2.8224	2.8224	NA	NA	NA	4.0642	4.0642
	Gable Roof Mansard Roof	1	-3.0837	-5.1744	-7.7877	-3.3451	-5.1744	-4.4406	-7.4511	-11.2143	-4.8169	-7.4511
		2	1.7771	1.7771	1.7771	3.0837	2.8224	2.5590	2.5590	2.5590	4.4406	4.0642
	Hip Roof	1	-2.8224	-4.9131	-7.2650	-3.3451	-5.1744	-4.0642	-7.0748	-10.4617	-4.8169	-7.4511
		2	1.7771	1.7771	1.7771	3.0837	2.8224	2.5590	2.5590	2.5590	4.4406	4.0642
	Monoslope Roof	1	-3.6064	-4.6517	-8.0490	-3.3451	-5.1744	-5.1932	-6.6985	-11.5906	-4.8169	-7.4511
		2	1.5157	1.5157	1.5157	3.0837	2.8224	2.1826	2.1826	2.1826	4.4406	4.0642
9	Flat Roof	1	-3.8908	-6.1071	-8.3234	-2.6596	-4.8759	-5.6028	-8.7943	-11.9858	-3.8298	-7.0212
		2	NA	NA	NA	2.6596	2.6596	NA	NA	NA	3.8298	3.8298
	Gable Roof Mansard Roof	1	-2.9058	-4.8759	-7.3384	-3.1521	-4.8759	-4.1844	-7.0212	-10.5673	-4.5390	-7.0212
		2	1.6745	1.6745	1.6745	2.9058	2.6596	2.4113	2.4113	2.4113	4.1844	3.8298
	Hip Roof	1	-2.6596	-4.6296	-6.8459	-3.1521	-4.8759	-3.8298	-6.6666	-9.8581	-4.5390	-7.0212
		2	1.6745	1.6745	1.6745	2.9058	2.6596	2.4113	2.4113	2.4113	4.1844	3.8298
	Monoslope Roof	1	-3.3983	-4.3834	-7.5847	-3.1521	-4.8759	-4.8936	-6.3120	-10.9219	-4.5390	-7.0212
		2	1.4283	1.4283	1.4283	2.9058	2.6596	2.0567	2.0567	2.0567	4.1844	3.8298
7.5	Flat Roof	1	-3.7320	-5.8579	-7.9837	-2.5510	-4.6768	-5.3741	-8.4353	-11.4965	-3.6735	-6.7347
		2	NA	NA	NA	2.5510	2.5510	NA	NA	NA	3.6735	3.6735
	Gable Roof Mansard Roof	1	-2.7872	-4.6768	-7.0389	-3.0234	-4.6768	-4.0136	-6.7347	-10.1360	-4.3537	-6.7347
		2	1.6062	1.6062	1.6062	2.7872	2.5510	2.3129	2.3129	2.3129	4.0136	3.6735
	Hip Roof	1	-2.5510	-4.4406	-6.5665	-3.0234	-4.6768	-3.6735	-6.3945	-9.4557	-4.3537	-6.7347
		2	1.6062	1.6062	1.6062	2.7872	2.5510	2.3129	2.3129	2.3129	4.0136	3.6735
	Monoslope Roof	1	-3.2596	-4.2044	-7.2751	-3.0234	-4.6768	-4.6939	-6.0544	-10.4761	-4.3537	-6.7347
		2	1.3700	1.3700	1.3700	2.7872	2.5510	1.9728	1.9728	1.9728	4.0136	3.6735
6	Flat Roof	1	-3.5732	-5.6086	-7.6440	-2.4425	-4.4778	-5.1454	-8.0764	-11.0073	-3.5171	-6.4481
		2	NA	NA	NA	2.4425	2.4425	NA	NA	NA	3.5171	3.5171
	Gable Roof Mansard Roof	1	-2.6686	-4.4778	-6.7394	-2.8948	-4.4778	-3.8428	-6.4481	-9.7047	-4.1685	-6.4481
		2	1.5378	1.5378	1.5378	2.6686	2.4425	2.2145	2.2145	2.2145	3.8428	3.5171
	Hip Roof	1	-2.4425	-4.2517	-6.2871	-2.8948	-4.4778	-3.5171	-6.1224	-9.0534	-4.1685	-6.4481
		2	1.5378	1.5378	1.5378	2.6686	2.4425	2.2145	2.2145	2.2145	3.8428	3.5171
	Monoslope Roof	1	-3.1209	-4.0255	-6.9655	-2.8948	-4.4778	-4.4941	-5.7968	-10.0303	-4.1685	-6.4481
		2	1.3117	1.3117	1.3117	2.6686	2.4425	1.8888	1.8888	1.8888	3.8428	3.5171
4.5	Flat Roof	1	-3.3747	-5.2970	-7.2193	-2.3068	-4.2291	-4.8596	-7.6277	-10.3958	-3.3217	-6.0899
		2	NA	NA	NA	2.3068	2.3068	NA	NA	NA	3.3217	3.3217
	Gable Roof Mansard Roof	1	-2.5204	-4.2291	-6.3650	-2.7339	-4.2291	-3.6293	-6.0899	-9.1655	-3.9369	-6.0899
		2	1.4524	1.4524	1.4524	2.5204	2.3068	2.0915	2.0915	2.0915	3.6293	3.3217
	Hip Roof	1	-2.3068	-4.0155	-5.9378	-2.7339	-4.2291	-3.3217	-5.7823	-8.5504	-3.9369	-6.0899
		2	1.4524	1.4524	1.4524	2.5204	2.3068	2.0915	2.0915	2.0915	3.6293	3.3217
	Monoslope Roof	1	-2.9475	-3.8019	-6.5785	-2.7339	-4.2291	-4.2444	-5.4747	-9.4731	-3.9369	-6.0899
		2	1.2388	1.2388	1.2388	2.5204	2.3068	1.7839	1.7839	1.7839	3.6293	3.3217

Table 207E.7-2
 Components and Cladding – Part 4
 C&C, $V = 350$ kph, $h = 4.5 - 15$ m
 Exposure C

h (m)	V (kph)		350				
	Roof Form	Load Case	Zone				
			1	2	3	4	5
15	Flat Roof	1	-8.4820	-13.3136	-18.1451	-5.7978	-10.6294
		2	NA	NA	NA	5.7978	5.7978
	Gable Roof Mansard Roof	1	-6.3347	-10.6294	-15.9978	-6.8715	-10.6294
		2	3.6505	3.6505	3.6505	6.3347	5.7978
	Hip Roof	1	-5.7978	-10.0925	-14.9241	-6.8715	-10.6294
		2	3.6505	3.6505	3.6505	6.3347	5.7978
12	Flat Roof	1	-7.4084	-9.5557	-16.5346	-6.8715	-10.6294
		2	3.1137	3.1137	3.1137	6.3347	5.7978
	Gable Roof Mansard Roof	1	-8.0929	-12.7029	-17.3128	-5.5319	-10.1418
		2	NA	NA	NA	5.5319	5.5319
	Hip Roof	1	-6.0441	-10.1418	-15.2639	-6.5563	-10.1418
		2	3.4830	3.4830	3.4830	6.0441	5.5319
9	Flat Roof	1	-5.5319	-9.6296	-14.2395	-6.5563	-10.1418
		2	3.4830	3.4830	3.4830	6.0441	5.5319
	Gable Roof Mansard Roof	1	-7.0685	-9.1174	-15.7761	-6.5563	-10.1418
		2	2.9708	2.9708	2.9708	6.0441	5.5319
	Hip Roof	1	-7.6260	-11.9700	-16.3139	-5.2127	-9.5567
		2	NA	NA	NA	5.2127	5.2127
7.5	Flat Roof	1	-5.6954	-9.5567	-14.3833	-6.1781	-9.5567
		2	3.2821	3.2821	3.2821	5.6954	5.2127
	Gable Roof Mansard Roof	1	-5.2127	-9.0740	-13.4180	-6.1781	-9.5567
		2	3.2821	3.2821	3.2821	5.6954	5.2127
	Hip Roof	1	-6.6607	-8.5914	-14.8660	-6.1781	-9.5567
		2	2.7994	2.7994	2.7994	5.6954	5.2127
6	Flat Roof	1	-7.3148	-11.4814	-15.6481	-5.0000	-9.1666
		2	NA	NA	NA	5.0000	5.0000
	Gable Roof Mansard Roof	1	-5.4629	-9.1666	-13.7962	-5.9259	-9.1666
		2	3.1481	3.1481	3.1481	5.4629	5.0000
	Hip Roof	1	-5.0000	-8.7037	-12.8703	-5.9259	-9.1666
		2	3.1481	3.1481	3.1481	5.4629	5.0000
4.5	Flat Roof	1	-6.3889	-8.2407	-14.2592	-5.9259	-9.1666
		2	2.6852	2.6852	2.6852	5.4629	5.0000
	Gable Roof Mansard Roof	1	-7.0035	-10.9929	-14.9822	-4.7872	-8.7766
		2	NA	NA	NA	4.7872	4.7872
	Hip Roof	1	-5.2305	-8.7766	-13.2092	-5.6737	-8.7766
		2	3.0142	3.0142	3.0142	5.2305	4.7872
4.5	Flat Roof	1	-4.7872	-8.3333	-12.3226	-5.6737	-8.7766
		2	3.0142	3.0142	3.0142	5.2305	4.7872
	Gable Roof Mansard Roof	1	-6.1170	-7.8900	-13.6524	-5.6737	-8.7766
		2	2.5709	2.5709	2.5709	5.2305	4.7872
	Hip Roof	1	-6.6144	-10.3821	-14.1498	-4.5213	-8.2890
		2	NA	NA	NA	4.5213	4.5213
4.5	Flat Roof	1	-4.9399	-8.2890	-12.4753	-5.3585	-8.2890
		2	2.8467	2.8467	2.8467	4.9399	4.5213
	Gable Roof Mansard Roof	1	-4.5213	-7.8703	-11.6380	-5.3585	-8.2890
		2	2.8467	2.8467	2.8467	4.9399	4.5213
	Hip Roof	1	-5.7772	-7.4517	-12.8939	-5.3585	-8.2890
		2	2.4281	2.4281	2.4281	4.9399	4.5213

Table 207E.7-2
Components and Cladding – Part 4
C&C, $V = 150$ -200 kph, $h = 18$ - 33 m
Exposure C

V (kph)			150					200				
h (m)	Roof Form	Load Case	Zone					Zone				
			1	2	3	4	5	1	2	3	4	5
33	Flat Roof	1	-1.8438	-2.8940	-3.9443	-1.2603	-2.3106	-3.2778	-5.1450	-7.0121	-2.2405	-4.1077
		2	NA	NA	NA	1.2603	1.2603	NA	NA	NA	2.2405	2.2405
	Gable Roof Mansard Roof	1	-1.3770	-2.3106	-3.4775	-1.4937	-2.3106	-2.4480	-4.1077	-6.1822	-2.6555	-4.1077
		2	0.7935	0.7935	0.7935	1.3770	1.2603	1.4107	1.4107	1.4107	2.4480	2.2405
	Hip Roof	1	-1.2603	-2.1939	-3.2441	-1.4937	-2.3106	-2.2405	-3.9002	-5.7673	-2.6555	-4.1077
		2	0.7935	0.7935	0.7935	1.3770	1.2603	1.4107	1.4107	1.4107	2.4480	2.2405
30	Monoslope Roof	1	-1.6104	-2.0772	-3.5942	-1.4937	-2.3106	-2.8629	-3.6927	-6.3897	-2.6555	-4.1077
		2	0.6768	0.6768	0.6768	1.3770	1.2603	1.2033	1.2033	1.2033	2.4480	2.2405
	Flat Roof	1	-1.8009	-2.8267	-3.8526	-1.2310	-2.2568	-3.2016	-5.0253	-6.8490	-2.1884	-4.0121
		2	NA	NA	NA	1.2310	1.2310	NA	NA	NA	2.1884	2.1884
	Gable Roof Mansard Roof	1	-1.3450	-2.2568	-3.3966	-1.4590	-2.2568	-2.3911	-4.0121	-6.0385	-2.5937	-4.0121
		2	0.7751	0.7751	0.7751	1.3450	1.2310	1.3779	1.3779	1.3779	2.3911	2.1884
27	Hip Roof	1	-1.2310	-2.1428	-3.1687	-1.4590	-2.2568	-2.1884	-3.8095	-5.6332	-2.5937	-4.0121
		2	0.7751	0.7751	0.7751	1.3450	1.2310	1.3779	1.3779	1.3779	2.3911	2.1884
	Monoslope Roof	1	-1.5729	-2.0289	-3.5106	-1.4590	-2.2568	-2.7963	-3.6069	-6.2411	-2.5937	-4.0121
		2	0.6611	0.6611	0.6611	1.3450	1.2310	1.1753	1.1753	1.1753	2.3911	2.1884
	Flat Roof	1	-1.7723	-2.7819	-3.7914	-1.2115	-2.2210	-3.1508	-4.9455	-6.7403	-2.1537	-3.9485
		2	NA	NA	NA	1.2115	1.2115	NA	NA	NA	2.1537	2.1537
24	Gable Roof Mansard Roof	1	-1.3236	-2.2210	-3.3427	-1.4358	-2.2210	-2.3531	-3.9485	-5.9426	-2.5525	-3.9485
		2	0.7628	0.7628	0.7628	1.3236	1.2115	1.3560	1.3560	1.3560	2.3531	2.1537
	Hip Roof	1	-1.2115	-2.1088	-3.1184	-1.4358	-2.2210	-2.1537	-3.7490	-5.5438	-2.5525	-3.9485
		2	0.7628	0.7628	0.7628	1.3236	1.2115	1.3560	1.3560	1.3560	2.3531	2.1537
	Monoslope Roof	1	-1.5480	-1.9967	-3.4549	-1.4358	-2.2210	-2.7520	-3.5496	-6.1420	-2.5525	-3.9485
		2	0.6506	0.6506	0.6506	1.3236	1.2115	1.1566	1.1566	1.1566	2.3531	2.1537
21	Flat Roof	1	-1.7294	-2.7146	-3.6997	-1.1821	-2.1673	-3.0746	-4.8259	-6.5772	-2.1016	-3.8529
		2	NA	NA	NA	1.1821	1.1821	NA	NA	NA	2.1016	2.1016
	Gable Roof Mansard Roof	1	-1.2916	-2.1673	-3.2619	-1.4011	-2.1673	-2.2962	-3.8529	-5.7988	-2.4908	-3.8529
		2	0.7443	0.7443	0.7443	1.2916	1.1821	1.3232	1.3232	1.3232	2.2962	2.1016
	Hip Roof	1	-1.1821	-2.0578	-3.0429	-1.4011	-2.1673	-2.1016	-3.6583	-5.4097	-2.4908	-3.8529
		2	0.7443	0.7443	0.7443	1.2916	1.1821	1.3232	1.3232	1.3232	2.2962	2.1016
18	Monoslope Roof	1	-1.5105	-1.9484	-3.3713	-1.4011	-2.1673	-2.6854	-3.4637	-5.9934	-2.4908	-3.8529
		2	0.6349	0.6349	0.6349	1.2916	1.1821	1.1286	1.1286	1.1286	2.2962	2.1016
	Flat Roof	1	-1.6723	-2.6248	-3.5774	-1.1431	-2.0956	-2.9729	-4.6664	-6.3598	-2.0321	-3.7256
		2	NA	NA	NA	1.1431	1.1431	NA	NA	NA	2.0321	2.0321
	Gable Roof Mansard Roof	1	-1.2489	-2.0956	-3.1540	-1.3547	-2.0956	-2.2203	-3.7256	-5.6072	-2.4084	-3.7256
		2	0.7197	0.7197	0.7197	1.2489	1.1431	1.2795	1.2795	1.2795	2.2203	2.0321
15	Hip Roof	1	-1.1431	-1.9898	-2.9423	-1.3547	-2.0956	-2.0321	-3.5374	-5.2308	-2.4084	-3.7256
		2	0.7197	0.7197	0.7197	1.2489	1.1431	1.2795	1.2795	1.2795	2.2203	2.0321
	Monoslope Roof	1	-1.4606	-1.8839	-3.2599	-1.3547	-2.0956	-2.5966	-3.3492	-5.7953	-2.4084	-3.7256
		2	0.6139	0.6139	0.6139	1.2489	1.1431	1.0913	1.0913	1.0913	2.2203	2.0321
	Flat Roof	1	-1.6151	-2.5351	-3.4551	-1.1040	-2.0240	-2.8713	-4.5068	-6.1424	-1.9626	-3.5982
		2	NA	NA	NA	1.1040	1.1040	NA	NA	NA	1.9626	1.9626
12	Gable Roof Mansard Roof	1	-1.2062	-2.0240	-3.0462	-1.3084	-2.0240	-2.1444	-3.5982	-5.4155	-2.3261	-3.5982
		2	0.6951	0.6951	0.6951	1.2062	1.1040	1.2357	1.2357	1.2357	2.1444	1.9626
	Hip Roof	1	-1.1040	-1.9218	-2.8417	-1.3084	-2.0240	-1.9626	-3.4165	-5.0520	-2.3261	-3.5982
		2	0.6951	0.6951	0.6951	1.2062	1.1040	1.2357	1.2357	1.2357	2.1444	1.9626
	Monoslope Roof	1	-1.4107	-1.8195	-3.1484	-1.3084	-2.0240	-2.5078	-3.2347	-5.5972	-2.3261	-3.5982
		2	0.5929	0.5929	0.5929	1.2062	1.1040	1.0540	1.0540	1.0540	2.1444	1.9626

Table 207E.7-2
 Components and Cladding – Part 4
 C&C, $V = 250$ -300 kph, $h = 18$ - 33 m
 Exposure C

V (kph)			250					300				
h (m)	Roof Form	Load Case	Zone					Zone				
			1	2	3	4	5	1	2	3	4	5
33	Flat Roof	1	-5.1216	-8.0390	-10.9564	-3.5009	-6.4182	-7.3751	-11.5761	-15.7772	-5.0412	-9.2422
		2	NA	NA	NA	3.5009	3.5009	NA	NA	NA	5.0412	5.0412
	Gable Roof Mansard Roof	1	-3.8250	-6.4182	-9.6598	-4.1492	-6.4182	-5.5080	-9.2422	-13.9100	-5.9748	-9.2422
		2	2.2042	2.2042	2.2042	3.8250	3.5009	3.1741	3.1741	3.1741	5.5080	5.0412
	Hip Roof	1	-3.5009	-6.0941	-9.0114	-4.1492	-6.4182	-5.0412	-8.7755	-12.9765	-5.9748	-9.2422
		2	2.2042	2.2042	2.2042	3.8250	3.5009	3.1741	3.1741	3.1741	5.5080	5.0412
	Monoslope Roof	1	-4.4733	-5.7699	-9.9839	-4.1492	-6.4182	-6.4416	-8.3087	-14.3768	-5.9748	-9.2422
		2	1.8801	1.8801	1.8801	3.8250	3.5009	2.7073	2.7073	2.7073	5.5080	5.0412
30	Flat Roof	1	-5.0025	-7.8520	-10.7016	-3.4194	-6.2690	-7.2036	-11.3069	-15.4103	-4.9240	-9.0273
		2	NA	NA	NA	3.4194	3.4194	NA	NA	NA	4.9240	4.9240
	Gable Roof Mansard Roof	1	-3.7360	-6.2690	-9.4351	-4.0527	-6.2690	-5.3799	-9.0273	-13.5866	-5.8358	-9.0273
		2	2.1530	2.1530	2.1530	3.7360	3.4194	3.1003	3.1003	3.1003	5.3799	4.9240
	Hip Roof	1	-3.4194	-5.9524	-8.8019	-4.0527	-6.2690	-4.9240	-8.5714	-12.6747	-5.8358	-9.0273
		2	2.1530	2.1530	2.1530	3.7360	3.4194	3.1003	3.1003	3.1003	5.3799	4.9240
	Monoslope Roof	1	-4.3693	-5.6357	-9.7517	-4.0527	-6.2690	-6.2918	-8.1155	-14.0425	-5.8358	-9.0273
		2	1.8364	1.8364	1.8364	3.7360	3.4194	2.6444	2.6444	2.6444	5.3799	4.9240
27	Flat Roof	1	-4.9231	-7.7274	-10.5317	-3.3652	-6.1695	-7.0893	-11.1275	-15.1656	-4.8458	-8.8840
		2	NA	NA	NA	3.3652	3.3652	NA	NA	NA	4.8458	4.8458
	Gable Roof Mansard Roof	1	-3.6767	-6.1695	-9.2853	-3.9883	-6.1695	-5.2945	-8.8840	-13.3709	-5.7432	-8.8840
		2	2.1188	2.1188	2.1188	3.6767	3.3652	3.0511	3.0511	3.0511	5.2945	4.8458
	Hip Roof	1	-3.3652	-5.8579	-8.6622	-3.9883	-6.1695	-4.8458	-8.4353	-12.4735	-5.7432	-8.8840
		2	2.1188	2.1188	2.1188	3.6767	3.3652	3.0511	3.0511	3.0511	5.2945	4.8458
	Monoslope Roof	1	-4.2999	-5.5463	-9.5969	-3.9883	-6.1695	-6.1919	-7.9866	-13.8196	-5.7432	-8.8840
		2	1.8072	1.8072	1.8072	3.6767	3.3652	2.6024	2.6024	2.6024	5.2945	4.8458
24	Flat Roof	1	-4.8040	-7.5404	-10.2769	-3.2837	-6.0202	-6.9178	-10.8582	-14.7987	-4.7286	-8.6691
		2	NA	NA	NA	3.2837	3.2837	NA	NA	NA	4.7286	4.7286
	Gable Roof Mansard Roof	1	-3.5878	-6.0202	-9.0607	-3.8918	-6.0202	-5.1664	-8.6691	-13.0474	-5.6043	-8.6691
		2	2.0675	2.0675	2.0675	3.5878	3.2837	2.9773	2.9773	2.9773	5.1664	4.7286
	Hip Roof	1	-3.2837	-5.7161	-8.4526	-3.8918	-6.0202	-4.7286	-8.2313	-12.1717	-5.6043	-8.6691
		2	2.0675	2.0675	2.0675	3.5878	3.2837	2.9773	2.9773	2.9773	5.1664	4.7286
	Monoslope Roof	1	-4.1959	-5.4121	-9.3647	-3.8918	-6.0202	-6.0421	-7.7934	-13.4852	-5.6043	-8.6691
		2	1.7635	1.7635	1.7635	3.5878	3.2837	2.5394	2.5394	2.5394	5.1664	4.7286
21	Flat Roof	1	-4.6452	-7.2912	-9.9372	-3.1752	-5.8212	-6.6891	-10.4993	-14.3095	-4.5723	-8.3825
		2	NA	NA	NA	3.1752	3.1752	NA	NA	NA	4.5723	4.5723
	Gable Roof Mansard Roof	1	-3.4692	-5.8212	-8.7612	-3.7632	-5.8212	-4.9956	-8.3825	-12.6161	-5.4190	-8.3825
		2	1.9992	1.9992	1.9992	3.4692	3.1752	2.8788	2.8788	2.8788	4.9956	4.5723
	Hip Roof	1	-3.1752	-5.5272	-8.1732	-3.7632	-5.8212	-4.5723	-7.9591	-11.7694	-5.4190	-8.3825
		2	1.9992	1.9992	1.9992	3.4692	3.1752	2.8788	2.8788	2.8788	4.9956	4.5723
	Monoslope Roof	1	-4.0572	-5.2332	-9.0552	-3.7632	-5.8212	-5.8423	-7.5358	-13.0394	-5.4190	-8.3825
		2	1.7052	1.7052	1.7052	3.4692	3.1752	2.4555	2.4555	2.4555	4.9956	4.5723
18	Flat Roof	1	-4.4864	-7.0419	-9.5974	-3.0666	-5.6222	-6.4604	-10.1403	-13.8203	-4.4160	-8.0959
		2	NA	NA	NA	3.0666	3.0666	NA	NA	NA	4.4160	4.4160
	Gable Roof Mansard Roof	1	-3.3506	-5.6222	-8.4616	-3.6345	-5.6222	-4.8248	-8.0959	-12.1848	-5.2337	-8.0959
		2	1.9308	1.9308	1.9308	3.3506	3.0666	2.7804	2.7804	2.7804	4.8248	4.4160
	Hip Roof	1	-3.0666	-5.3382	-7.8937	-3.6345	-5.6222	-4.4160	-7.6870	-11.3670	-5.2337	-8.0959
		2	1.9308	1.9308	1.9308	3.3506	3.0666	2.7804	2.7804	2.7804	4.8248	4.4160
	Monoslope Roof	1	-3.9185	-5.0543	-8.7456	-3.6345	-5.6222	-5.6426	-7.2782	-12.5937	-5.2337	-8.0959
		2	1.6469	1.6469	1.6469	3.3506	3.0666	2.3715	2.3715	2.3715	4.8248	4.4160

Table 207E.7-2
Components and Cladding – Part 4
C&C, $V = 350$ kph, $h = 18 - 33$ m
Exposure C

h (m)	V (kph)		350				
	Roof Form	Load Case	Zone				
			1	2	3	4	5
33	Flat Roof	1	-10.0384	-15.7564	-21.4745	-6.8617	-12.5797
		2	NA	NA	NA	6.8617	6.8617
	Gable Roof Mansard Roof	1	-7.4970	-12.5797	-18.9331	-8.1323	-12.5797
		2	4.3203	4.3203	4.3203	7.4970	6.8617
	Hip Roof	1	-6.8617	-11.9444	-17.6624	-8.1323	-12.5797
		2	4.3203	4.3203	4.3203	7.4970	6.8617
30	Flat Roof	1	-8.7677	-11.3090	-19.5685	-8.1323	-12.5797
		2	3.6850	3.6850	3.6850	7.4970	6.8617
	Flat Roof	1	-9.8049	-15.3900	-20.9751	-6.7021	-12.2872
		2	NA	NA	NA	6.7021	6.7021
	Gable Roof Mansard Roof	1	-7.3227	-12.2872	-18.4928	-7.9432	-12.2872
		2	4.2198	4.2198	4.2198	7.3227	6.7021
27	Hip Roof	1	-6.7021	-11.6666	-17.2517	-7.9432	-12.2872
		2	4.2198	4.2198	4.2198	7.3227	6.7021
	Monoslope Roof	1	-8.5638	-11.0460	-19.1134	-7.9432	-12.2872
		2	3.5993	3.5993	3.5993	7.3227	6.7021
	Flat Roof	1	-9.6493	-15.1457	-20.6421	-6.5957	-12.0921
		2	NA	NA	NA	6.5957	6.5957
24	Gable Roof Mansard Roof	1	-7.2064	-12.0921	-18.1993	-7.8171	-12.0921
		2	4.1529	4.1529	4.1529	7.2064	6.5957
	Hip Roof	1	-6.5957	-11.4814	-16.9778	-7.8171	-12.0921
		2	4.1529	4.1529	4.1529	7.2064	6.5957
	Monoslope Roof	1	-8.4279	-10.8707	-18.8100	-7.8171	-12.0921
		2	3.5421	3.5421	3.5421	7.2064	6.5957
21	Flat Roof	1	-9.4158	-14.7793	-20.1427	-6.4361	-11.7996
		2	NA	NA	NA	6.4361	6.4361
	Gable Roof Mansard Roof	1	-7.0321	-11.7996	-17.7590	-7.6280	-11.7996
		2	4.0524	4.0524	4.0524	7.0321	6.4361
	Hip Roof	1	-6.4361	-11.2036	-16.5671	-7.6280	-11.7996
		2	4.0524	4.0524	4.0524	7.0321	6.4361
18	Monoslope Roof	1	-8.2240	-10.6077	-18.3549	-7.6280	-11.7996
		2	3.4564	3.4564	3.4564	7.0321	6.4361
	Flat Roof	1	-9.1046	-14.2907	-19.4768	-6.2234	-11.4095
		2	NA	NA	NA	6.2234	6.2234
	Gable Roof Mansard Roof	1	-6.7996	-11.4095	-17.1719	-7.3758	-11.4095
		2	3.9184	3.9184	3.9184	6.7996	6.2234
18	Hip Roof	1	-6.2234	-10.8333	-16.0194	-7.3758	-11.4095
		2	3.9184	3.9184	3.9184	6.7996	6.2234
	Monoslope Roof	1	-7.9521	-10.2570	-17.7481	-7.3758	-11.4095
		2	3.3422	3.3422	3.3422	6.7996	6.2234
	Flat Roof	1	-8.7933	-13.8021	-18.8110	-6.0106	-11.0194
		2	NA	NA	NA	6.0106	6.0106
18	Gable Roof Mansard Roof	1	-6.5671	-11.0194	-16.5848	-7.1237	-11.0194
		2	3.7845	3.7845	3.7845	6.5671	6.0106
	Hip Roof	1	-6.0106	-10.4629	-15.4717	-7.1237	-11.0194
		2	3.7845	3.7845	3.7845	6.5671	6.0106
	Monoslope Roof	1	-7.6802	-9.9064	-17.1414	-7.1237	-11.0194
		2	3.2279	3.2279	3.2279	6.5671	6.0106

Table 207E.7-2
Components and Cladding – Part 4
C&C, $V = 150$ -200 kph, $h = 36$ - 48 m
Exposure C

V (kph)			150					200				
h (m)	Roof Form	Load Case	Zone					Zone				
			1	2	3	4	5	1	2	3	4	5
48	Flat Roof	1	-1.9867	-3.1184	-4.2501	-1.3580	-2.4897	-3.5319	-5.5438	-7.5556	-2.4142	-4.4261
		2	NA	NA	NA	1.3580	1.3580	NA	NA	NA	2.4142	2.4142
	Gable Roof Mansard Roof	1	-1.4837	-2.4897	-3.7471	-1.6095	-2.4897	-2.6378	-4.4261	-6.6615	-2.8613	-4.4261
		2	0.8550	0.8550	0.8550	1.4837	1.3580	1.5201	1.5201	1.5201	2.6378	2.4142
	Hip Roof	1	-1.3580	-2.3639	-3.4956	-1.6095	-2.4897	-2.4142	-4.2025	-6.2144	-2.8613	-4.4261
		2	0.8550	0.8550	0.8550	1.4837	1.3580	1.5201	1.5201	1.5201	2.6378	2.4142
	Monoslope Roof	1	-1.7352	-2.2382	-3.8728	-1.6095	-2.4897	-3.0848	-3.9790	-6.8850	-2.8613	-4.4261
		2	0.7293	0.7293	0.7293	1.4837	1.3580	1.2965	1.2965	1.2965	2.6378	2.4142
45	Flat Roof	1	-1.9724	-3.0959	-4.2195	-1.3482	-2.4718	-3.5065	-5.5039	-7.5013	-2.3969	-4.3942
		2	NA	NA	NA	1.3482	1.3482	NA	NA	NA	2.3969	2.3969
	Gable Roof Mansard Roof	1	-1.4731	-2.4718	-3.7201	-1.5979	-2.4718	-2.6188	-4.3942	-6.6136	-2.8407	-4.3942
		2	0.8489	0.8489	0.8489	1.4731	1.3482	1.5091	1.5091	1.5091	2.6188	2.3969
	Hip Roof	1	-1.3482	-2.3469	-3.4705	-1.5979	-2.4718	-2.3969	-4.1723	-6.1697	-2.8407	-4.3942
		2	0.8489	0.8489	0.8489	1.4731	1.3482	1.5091	1.5091	1.5091	2.6188	2.3969
	Monoslope Roof	1	-1.7227	-2.2221	-3.8450	-1.5979	-2.4718	-3.0627	-3.9504	-6.8355	-2.8407	-4.3942
		2	0.7241	0.7241	0.7241	1.4731	1.3482	1.2872	1.2872	1.2872	2.6188	2.3969
42	Flat Roof	1	-1.9438	-3.0511	-4.1583	-1.3287	-2.4359	-3.4557	-5.4241	-7.3926	-2.3621	-4.3306
		2	NA	NA	NA	1.3287	1.3287	NA	NA	NA	2.3621	2.3621
	Gable Roof Mansard Roof	1	-1.4517	-2.4359	-3.6662	-1.5747	-2.4359	-2.5808	-4.3306	-6.5177	-2.7996	-4.3306
		2	0.8366	0.8366	0.8366	1.4517	1.3287	1.4873	1.4873	1.4873	2.5808	2.3621
	Hip Roof	1	-1.3287	-2.3129	-3.4202	-1.5747	-2.4359	-2.3621	-4.1118	-6.0803	-2.7996	-4.3306
		2	0.8366	0.8366	0.8366	1.4517	1.3287	1.4873	1.4873	1.4873	2.5808	2.3621
	Monoslope Roof	1	-1.6978	-2.1899	-3.7892	-1.5747	-2.4359	-3.0183	-3.8931	-6.7364	-2.7996	-4.3306
		2	0.7136	0.7136	0.7136	1.4517	1.3287	1.2685	1.2685	1.2685	2.5808	2.3621
39	Flat Roof	1	-1.9152	-3.0062	-4.0972	-1.3092	-2.4001	-3.4049	-5.3444	-7.2839	-2.3274	-4.2669
		2	NA	NA	NA	1.3092	1.3092	NA	NA	NA	2.3274	2.3274
	Gable Roof Mansard Roof	1	-1.4304	-2.4001	-3.6123	-1.5516	-2.4001	-2.5429	-4.2669	-6.4219	-2.7584	-4.2669
		2	0.8243	0.8243	0.8243	1.4304	1.3092	1.4654	1.4654	1.4654	2.5429	2.3274
	Hip Roof	1	-1.3092	-2.2789	-3.3699	-1.5516	-2.4001	-2.3274	-4.0514	-5.9909	-2.7584	-4.2669
		2	0.8243	0.8243	0.8243	1.4304	1.3092	1.4654	1.4654	1.4654	2.5429	2.3274
	Monoslope Roof	1	-1.6728	-2.1577	-3.7335	-1.5516	-2.4001	-2.9739	-3.8359	-6.6374	-2.7584	-4.2669
		2	0.7031	0.7031	0.7031	1.4304	1.3092	1.2499	1.2499	1.2499	2.5429	2.3274
36	Flat Roof	1	-1.8724	-2.9389	-4.0054	-1.2798	-2.3464	-3.3287	-5.2247	-7.1208	-2.2753	-4.1713
		2	NA	NA	NA	1.2798	1.2798	NA	NA	NA	2.2753	2.2753
	Gable Roof Mansard Roof	1	-1.3984	-2.3464	-3.5314	-1.5169	-2.3464	-2.4860	-4.1713	-6.2781	-2.6966	-4.1713
		2	0.8058	0.8058	0.8058	1.3984	1.2798	1.4326	1.4326	1.4326	2.4860	2.2753
	Hip Roof	1	-1.2798	-2.2279	-3.2944	-1.5169	-2.3464	-2.2753	-3.9607	-5.8567	-2.6966	-4.1713
		2	0.8058	0.8058	0.8058	1.3984	1.2798	1.4326	1.4326	1.4326	2.4860	2.2753
	Monoslope Roof	1	-1.6354	-2.1094	-3.6499	-1.5169	-2.3464	-2.9073	-3.7500	-6.4888	-2.6966	-4.1713
		2	0.6873	0.6873	0.6873	1.3984	1.2798	1.2219	1.2219	1.2219	2.4860	2.2753

Table 207E.7-2
Components and Cladding – Part 4
C&C, $V = 250$ -300 kph, $h = 36$ - 48 m
Exposure C

V (kph)			250					300				
h (m)	Roof Form	Load Case	Zone					Zone				
			1	2	3	4	5	1	2	3	4	5
48	Flat Roof	1	-5.5186	-8.6622	-11.8057	-3.7722	-6.9158	-7.9468	-12.4735	-17.0002	-5.4320	-9.9587
		2	NA	NA	NA	3.7722	3.7722	NA	NA	NA	5.4320	5.4320
	Gable Roof Mansard Roof	1	-4.1215	-6.9158	-10.4086	-4.4708	-6.9158	-5.9350	-9.9587	-14.9883	-6.4379	-9.9587
		2	2.3751	2.3751	2.3751	4.1215	3.7722	3.4202	3.4202	3.4202	5.9350	5.4320
	Hip Roof	1	-3.7722	-6.5665	-9.7100	-4.4708	-6.9158	-5.4320	-9.4557	-13.9824	-6.4379	-9.9587
		2	2.3751	2.3751	2.3751	4.1215	3.7722	3.4202	3.4202	3.4202	5.9350	5.4320
	Monoslope Roof	1	-4.8201	-6.2172	-10.7579	-4.4708	-6.9158	-6.9409	-8.9528	-15.4913	-6.4379	-9.9587
		2	2.0258	2.0258	2.0258	4.1215	3.7722	2.9172	2.9172	2.9172	5.9350	5.4320
45	Flat Roof	1	-5.4789	-8.5998	-11.7208	-3.7451	-6.8660	-7.8897	-12.3838	-16.8779	-5.3929	-9.8871
		2	NA	NA	NA	3.7451	3.7451	NA	NA	NA	5.3929	5.3929
	Gable Roof Mansard Roof	1	-4.0919	-6.8660	-10.3337	-4.4386	-6.8660	-5.8923	-9.8871	-14.8805	-6.3916	-9.8871
		2	2.3580	2.3580	2.3580	4.0919	3.7451	3.3956	3.3956	3.3956	5.8923	5.3929
	Hip Roof	1	-3.7451	-6.5192	-9.6402	-4.4386	-6.8660	-5.3929	-9.3877	-13.8818	-6.3916	-9.8871
		2	2.3580	2.3580	2.3580	4.0919	3.7451	3.3956	3.3956	3.3956	5.8923	5.3929
	Monoslope Roof	1	-4.7854	-6.1725	-10.6805	-4.4386	-6.8660	-6.8910	-8.8884	-15.3799	-6.3916	-9.8871
		2	2.0113	2.0113	2.0113	4.0919	3.7451	2.8962	2.8962	2.8962	5.8923	5.3929
42	Flat Roof	1	-5.3995	-8.4752	-11.5509	-3.6908	-6.7665	-7.7753	-12.2043	-16.6333	-5.3148	-9.7438
		2	NA	NA	NA	3.6908	3.6908	NA	NA	NA	5.3148	5.3148
	Gable Roof Mansard Roof	1	-4.0326	-6.7665	-10.1839	-4.3743	-6.7665	-5.8069	-9.7438	-14.6649	-6.2990	-9.7438
		2	2.3238	2.3238	2.3238	4.0326	3.6908	3.3463	3.3463	3.3463	5.8069	5.3148
	Hip Roof	1	-3.6908	-6.4248	-9.5004	-4.3743	-6.7665	-5.3148	-9.2517	-13.6806	-6.2990	-9.7438
		2	2.3238	2.3238	2.3238	4.0326	3.6908	3.3463	3.3463	3.3463	5.8069	5.3148
	Monoslope Roof	1	-4.7160	-6.0830	-10.5257	-4.3743	-6.7665	-6.7911	-8.7595	-15.1570	-6.2990	-9.7438
		2	1.9821	1.9821	1.9821	4.0326	3.6908	2.8542	2.8542	2.8542	5.8069	5.3148
39	Flat Roof	1	-5.3201	-8.3506	-11.3810	-3.6365	-6.6670	-7.6610	-12.0248	-16.3887	-5.2366	-9.6005
		2	NA	NA	NA	3.6365	3.6365	NA	NA	NA	5.2366	5.2366
	Gable Roof Mansard Roof	1	-3.9733	-6.6670	-10.0342	-4.3100	-6.6670	-5.7215	-9.6005	-14.4492	-6.2064	-9.6005
		2	2.2897	2.2897	2.2897	3.9733	3.6365	3.2971	3.2971	3.2971	5.7215	5.2366
	Hip Roof	1	-3.6365	-6.3303	-9.3607	-4.3100	-6.6670	-5.2366	-9.1156	-13.4794	-6.2064	-9.6005
		2	2.2897	2.2897	2.2897	3.9733	3.6365	3.2971	3.2971	3.2971	5.7215	5.2366
	Monoslope Roof	1	-4.6467	-5.9936	-10.3709	-4.3100	-6.6670	-6.6912	-8.6307	-14.9341	-6.2064	-9.6005
		2	1.9530	1.9530	1.9530	3.9733	3.6365	2.8123	2.8123	2.8123	5.7215	5.2366
36	Flat Roof	1	-5.2010	-8.1636	-11.1262	-3.5551	-6.5177	-7.4895	-11.7556	-16.0218	-5.1194	-9.3855
		2	NA	NA	NA	3.5551	3.5551	NA	NA	NA	5.1194	5.1194
	Gable Roof Mansard Roof	1	-3.8843	-6.5177	-9.8095	-4.2135	-6.5177	-5.5934	-9.3855	-14.1257	-6.0674	-9.3855
		2	2.2384	2.2384	2.2384	3.8843	3.5551	3.2233	3.2233	3.2233	5.5934	5.1194
	Hip Roof	1	-3.5551	-6.1886	-9.1512	-4.2135	-6.5177	-5.1194	-8.9115	-13.1777	-6.0674	-9.3855
		2	2.2384	2.2384	2.2384	3.8843	3.5551	3.2233	3.2233	3.2233	5.5934	5.1194
	Monoslope Roof	1	-4.5427	-5.8594	-10.1387	-4.2135	-6.5177	-6.5414	-8.4375	-14.5997	-6.0674	-9.3855
		2	1.9092	1.9092	1.9092	3.8843	3.5551	2.7493	2.7493	2.7493	5.5934	5.1194

Table 207E.7-2
 Components and Cladding – Part 4
 C&C, $V = 350$ kph, $h = 36 - 48$ m
 Exposure C

V (kph)			350				
h (m)	Roof Form	Load Case	Zone				
			1	2	3	4	5
48	Flat Roof	1	-10.8165	-16.9778	-23.1392	-7.3936	-13.5549
		2	NA	NA	NA	7.3936	7.3936
	Gable Roof Mansard Roof	1	-8.0782	-13.5549	-20.4008	-8.7628	-13.5549
		2	4.6552	4.6552	4.6552	8.0782	7.3936
	Hip Roof	1	-7.3936	-12.8703	-19.0316	-8.7628	-13.5549
		2	4.6552	4.6552	4.6552	8.0782	7.3936
	Monoslope Roof	1	-9.4474	-12.1857	-21.0854	-8.7628	-13.5549
		2	3.9706	3.9706	3.9706	8.0782	7.3936
45	Flat Roof	1	-10.7387	-16.8557	-22.9727	-7.3404	-13.4574
		2	NA	NA	NA	7.3404	7.3404
	Gable Roof Mansard Roof	1	-8.0201	-13.4574	-20.2540	-8.6997	-13.4574
		2	4.6217	4.6217	4.6217	8.0201	7.3404
	Hip Roof	1	-7.3404	-12.7777	-18.8947	-8.6997	-13.4574
		2	4.6217	4.6217	4.6217	8.0201	7.3404
	Monoslope Roof	1	-9.3794	-12.0980	-20.9337	-8.6997	-13.4574
		2	3.9421	3.9421	3.9421	8.0201	7.3404
42	Flat Roof	1	-10.5831	-16.6114	-22.6398	-7.2340	-13.2623
		2	NA	NA	NA	7.2340	7.2340
	Gable Roof Mansard Roof	1	-7.9038	-13.2623	-19.9605	-8.5736	-13.2623
		2	4.5547	4.5547	4.5547	7.9038	7.2340
	Hip Roof	1	-7.2340	-12.5925	-18.6209	-8.5736	-13.2623
		2	4.5547	4.5547	4.5547	7.9038	7.2340
	Monoslope Roof	1	-9.2435	-11.9227	-20.6303	-8.5736	-13.2623
		2	3.8849	3.8849	3.8849	7.9038	7.2340
39	Flat Roof	1	-10.4274	-16.3671	-22.3068	-7.1276	-13.0673
		2	NA	NA	NA	7.1276	7.1276
	Gable Roof Mansard Roof	1	-7.7876	-13.0673	-19.6670	-8.4476	-13.0673
		2	4.4878	4.4878	4.4878	7.7876	7.1276
	Hip Roof	1	-7.1276	-12.4073	-18.3470	-8.4476	-13.0673
		2	4.4878	4.4878	4.4878	7.7876	7.1276
	Monoslope Roof	1	-9.1075	-11.7474	-20.3269	-8.4476	-13.0673
		2	3.8278	3.8278	3.8278	7.7876	7.1276
36	Flat Roof	1	-10.1940	-16.0007	-21.8074	-6.9680	-12.7748
		2	NA	NA	NA	6.9680	6.9680
	Gable Roof Mansard Roof	1	-7.6132	-12.7748	-19.2267	-8.2584	-12.7748
		2	4.3873	4.3873	4.3873	7.6132	6.9680
	Hip Roof	1	-6.9680	-12.1296	-17.9363	-8.2584	-12.7748
		2	4.3873	4.3873	4.3873	7.6132	6.9680
	Monoslope Roof	1	-8.9036	-11.4844	-19.8718	-8.2584	-12.7748
		2	3.7421	3.7421	3.7421	7.6132	6.9680

Part 5: Open Buildings

Commentary:

In determining loads on component and cladding elements for open building roofs using Figures 207E.8-1, 207E.8-2 and 207E.8-3, it is important for the designer to note that the net pressure coefficient C_N is based on contributions from the top and bottom surfaces of the roof. This implies that the element receives load from both surfaces. Such would not be the case if the surface below the roof were separated structurally from the top roof surface. In this case, the pressure coefficient should be separated for the effect of top and bottom pressures, or conservatively, each surface could be designed using the C_N value from Figures 207E.8-1, 207E.8-2 and 207E.8-3.

207E.8 Building Types

The provisions of Section 207E.8 are applicable to an open building of all heights having a pitched free roof, monosloped free roof, or troughed free roof. The steps required for the determination of wind loads on components and cladding for these building types is shown in Table 207E.8-1.

207E.8.1 Conditions

For the determination of the design wind pressures on components and claddings using the provisions of Section 207E.8.2, the conditions indicated on the selected figure(s) shall be applicable to the building under consideration.

207E.8.2 Design Wind Pressures

The net design wind pressure for component and cladding elements of open buildings of all heights with monoslope, pitched, and troughed roofs shall be determined by the following equation:

$$p = q_h G C_N \quad (207E.8-1)$$

where

- q_h = velocity pressure evaluated at mean roof height h using the exposure as defined in Section 207A.7.3 that results in the highest wind loads for any wind direction at the site
- G = gust-effect factor from Section 207A.9
- C_N = net pressure coefficient given in:
 - Figure 207E.8-1 for monosloped roof
 - Figure 207E.8-2 for pitched roof
 - Figure 207E.8-3 for troughed roof

Net pressure coefficients C_N include contributions from top and bottom surfaces. All load cases shown for each roof angle shall be investigated. Plus and minus signs signify pressure acting toward and away from the top surface of the roof, respectively.

User Note:

Use Part 5 of Section 207E for determining wind pressures for C&C of open buildings having pitched, monoslope or troughed roofs. These provisions are based on the Directional Procedure with wind pressures calculated from the specified equation applicable to each roof surface.

Table 207E.8-1
Steps to Determine C&C Wind Loads
Open Buildings

- | | |
|---------|--|
| Step 1: | Determine risk category, see Table 103-1 |
| Step 2: | Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C |
| Step 3: | Determine wind load parameters: <ul style="list-style-type: none"> ➤ Wind directionality factor K_{zt}, see Section 207A.6 and Table 207A.6-1 ➤ Exposure category B, C or D, see Section 207A.7 ➤ Topographic factor K_{zt}, see Section 207A.8 and Figure 207A.8-1 ➤ Gust effect factor, G, see Section 207A.9 |
| Step 4: | Determine velocity pressure exposure coefficient, K_z or K_h , see Table 207E.3-1 |
| Step 5: | Determine velocity pressure, q_h , see Equation 207E.3-1 |
| Step 6: | Determine net pressure coefficients, C_N <ul style="list-style-type: none"> ➤ Monosloped roof, see Figure 207E.8-1 ➤ Pitched roof, see Figure 207E.8-2 ➤ Troughed roof, see Figure 207E.8-3 |
| Step 7: | Calculate wind pressure, p , see Equation 207E.8-1 |

Part 6: Building Appurtenances and Rooftop Structures and Equipment

207E.9 Parapets

The design wind pressure for component and cladding elements of parapets for all building types and heights, except enclosed buildings with $h \leq 48$ m for which the provisions of Part 4 are used, shall be determined from the following equation:

$$p = q_p[(GC_p) - (GC_{pi})] \quad (207E.9-1)$$

where

- q_p = velocity pressure evaluated at the top of the parapet
- (GC_p) = external pressure coefficient given in
- Figure 207E.4-1 for walls with $h > 18$ m
 - Figures 207E.4-2A to 207E.4-2C for flat roofs, gable roofs, and hip roofs
 - Figure 207E.4-3 for stepped roofs
 - Figure 207E.4-4 for multispans gable roofs
 - Figure 207E.4-5A and 207E.4-5B for monoslope roofs
 - Figure 207E.4-6 for sawtooth roofs
 - Figure 207E.4-7 for domed roofs of all heights
 - Figure 207E.6-1 for walls and flat roofs with $h > 18$ m
 - Figure 207B.4-3 for footnote 4 for arched roofs
- (GC_{pi}) = internal pressure coefficient from Table 207A.11-1, based on the porosity of the parapet envelope

Two load cases, see Figure 207E.9-1, shall be considered:

- Load Case A: Windward Parapet shall consist of applying the applicable positive wall pressure from Figure 207E.4-1 ($h \leq 18$ m) or Figure 207E.6-1 ($h > 18$ m) to the windward surface of the parapet while applying the applicable negative edge or corner zone roof pressure from Figures 207E.4-2 (A, B or C), 207E.4-3, 207E.4-4, 207E.4-5 (A or B), 207E.4-6, 207E.4-7, Figure 207B.4-3 footnote 4, or Figure 207E.6-1 ($h > 18$ m) as applicable to the leeward surface of the parapet.

- Load Case B: Leeward Parapet shall consist of applying the applicable positive wall pressure from Figure 207E.4-1 ($h \leq 18$ m) or Figure 207E.6-1 ($h > 18$ m) to the windward surface of the parapet, and applying the applicable negative wall pressure from Figure 207E.4-1 ($h \leq 18$ m) or Figure 207E.6-1 ($h > 18$ m) as applicable to the leeward surface. Edge and corner zones shall be arranged as shown in the applicable figures. (GC_p) shall be determined for appropriate roof angle and effective wind area from the applicable figures.

If internal pressure is present, both load cases should be evaluated under positive and negative internal pressure.

The steps required for the determination of wind loads on component and cladding of parapets are shown in Table 207E.9-1.

User Note:

Use Part 6 of Section 207E for determining wind pressures for C&C on roof overhangs and parapets of buildings. These provisions are based on the Directional Procedure with wind pressures calculated from the specified equation applicable to each roof overhang or parapet surface.

Table 207E.9-1
Steps to Determine C&C Wind Loads
Parapets

Step 1:	Determine risk category, see Table 103-1
Step 2:	Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C
Step 3:	Determine wind load parameters: ➤ Wind directionality factor K_{zt}, see Section 207A.6 and Table 207A.6-1 ➤ Exposure category B, C or D, see Section 207A.7 ➤ Topographic factor K_{zt}, see Section 207A.8 and Figure 207A.8-1 ➤ Enclosure classification, see Section 207A.10 ➤ Internal pressure coefficient, (GC_{pi}), see Section 207A.11 and Table 207A.11-1
Step 4:	Determine velocity pressure exposure coefficient, K_h , at the top of the parapet see Table 207E.3-1
Step 5:	Determine velocity pressure, q_p , at the top of the parapet, see Equation 207E.3-1
Step 6:	Determine external pressure coefficient for wall and roof surfaces adjacent to parapet, (GC_p) ➤ Walls with $h \leq 18$ m, see Figure 207E.4-1 ➤ Flat, gable and hip roofs, see Figures 207E.4-2A to 207E.4-2C ➤ Stepped roofs, see Figure 207E.4-3 ➤ Multispan gable roofs, see Figure 207E.4-4 ➤ Monoslope roofs, Figures 207E.4-5A and 207E.4-5B ➤ Sawtooth roofs, see Figure 207E.4-6 ➤ Domed roofs of all heights, see Figure 207E.4-7 ➤ Walls and flat roofs with $h > 18$ m, see Figure 207E.6-1 ➤ Arched roofs, see footnote 4 of Figure 207B.4-3
Step 7:	Calculate wind pressure, p , see Equation 207E.9-1 on windward and leeward face of parapet, considering two load cases (Case A and Case B) as shown in Figure 207E.9-1

207E.10 Roof Overhangs

The design wind pressure for roof overhangs of enclosed and partially enclosed buildings of all heights, except enclosed buildings with $h \leq 48$ m for which the provisions of Part 4 are used, shall be determined from the following equation:

$$p = q_h[(GC_p) - (GC_{pi})] \text{ (N/m}^2\text{)} \quad (207E.10-1)$$

where

q_h = velocity pressure from Section 207E.3.2 evaluated at mean roof height h using exposure defined in Section 207A.7.3

(GC_p) = external pressure coefficients for overhangs given in Figures 207E.4-2A to 207E.4-2C (flat roofs, gable roofs, and hip roofs), including contributions from top and bottom surfaces of overhang. The external pressure coefficient for the covering on the underside of the roof overhang is the same as the external pressure coefficient on the adjacent wall surface, adjusted for effective wind area, determined from Figure 207E.4-1 or Figure 207E.6-1 as applicable

(GC_{pi}) internal pressure coefficient given in Table 207A.11-1

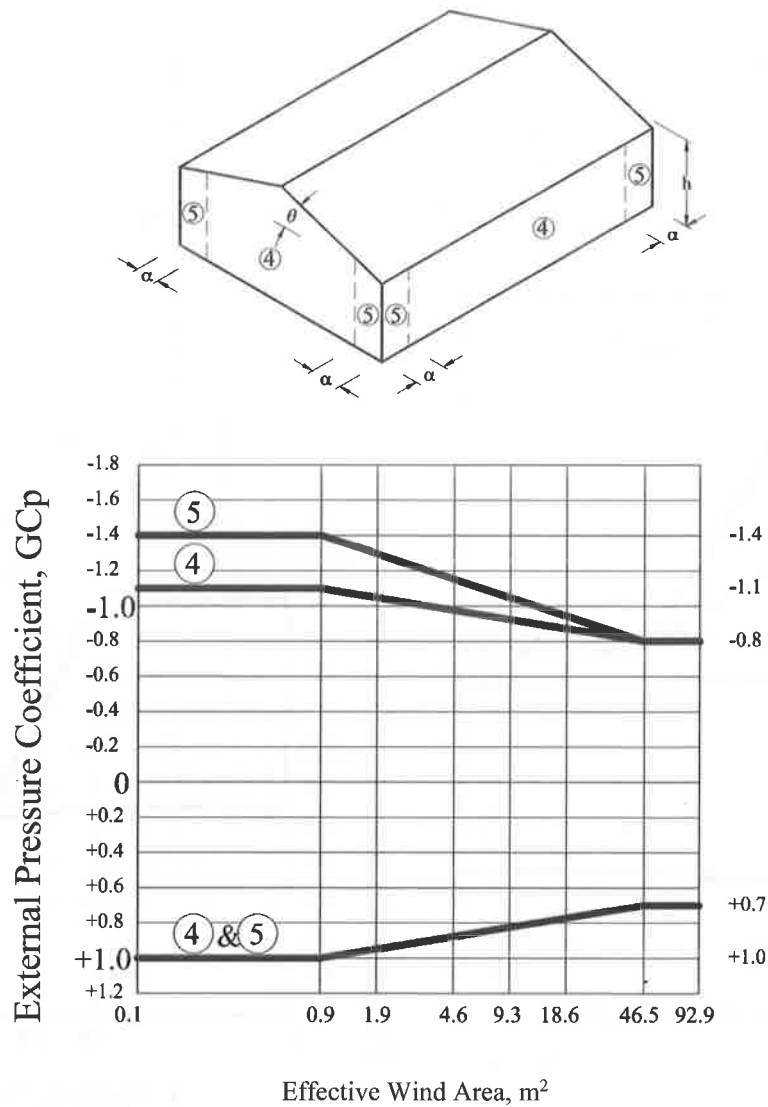
The steps required for the determination of wind loads on components and cladding of roof overhangs are shown in Table 207E.10-1.

Table 207E.10-1
Steps to Determine C&C Wind Loads
Roof Overhangs

- | | |
|---------|---|
| Step 1: | Determine risk category, see Table 103-1 |
| Step 2: | Determine the basic wind speed, V , for the applicable risk category, see Figure 207A.5-1A, B or C |
| Step 3: | Determine wind load parameters: <ul style="list-style-type: none"> ➤ Wind directionality factor K_{zt}, see Section 207A.6 and Table 207A.6-1 ➤ Exposure category B, C or D, see Section 207A.7 ➤ Topographic factor K_{zt}, see Section 207A.8 and Figure 207A.8-1 ➤ Enclosure classification, see Section 207A.10 ➤ Internal pressure coefficient, (GC_{pi}), see Section 207A.11 and Table 207A.11-1 |
| Step 4: | Determine velocity pressure exposure coefficient, K_h , see Table 207E.3-1 |
| Step 5: | Determine velocity pressure, q_h , at mean roof height h using Equation 207E.3-1 |
| Step 6: | Determine external pressure coefficient, (GC_p) , using Figures 207E.4-2A through C for flat, gabled and hip roofs |
| Step 7: | Calculate wind pressure, p , using Equation 207E.10-1, refer to Figure 207E.10-1 |

207E.11 Rooftop Structures and Equipment for Buildings with $h \leq 18$ m

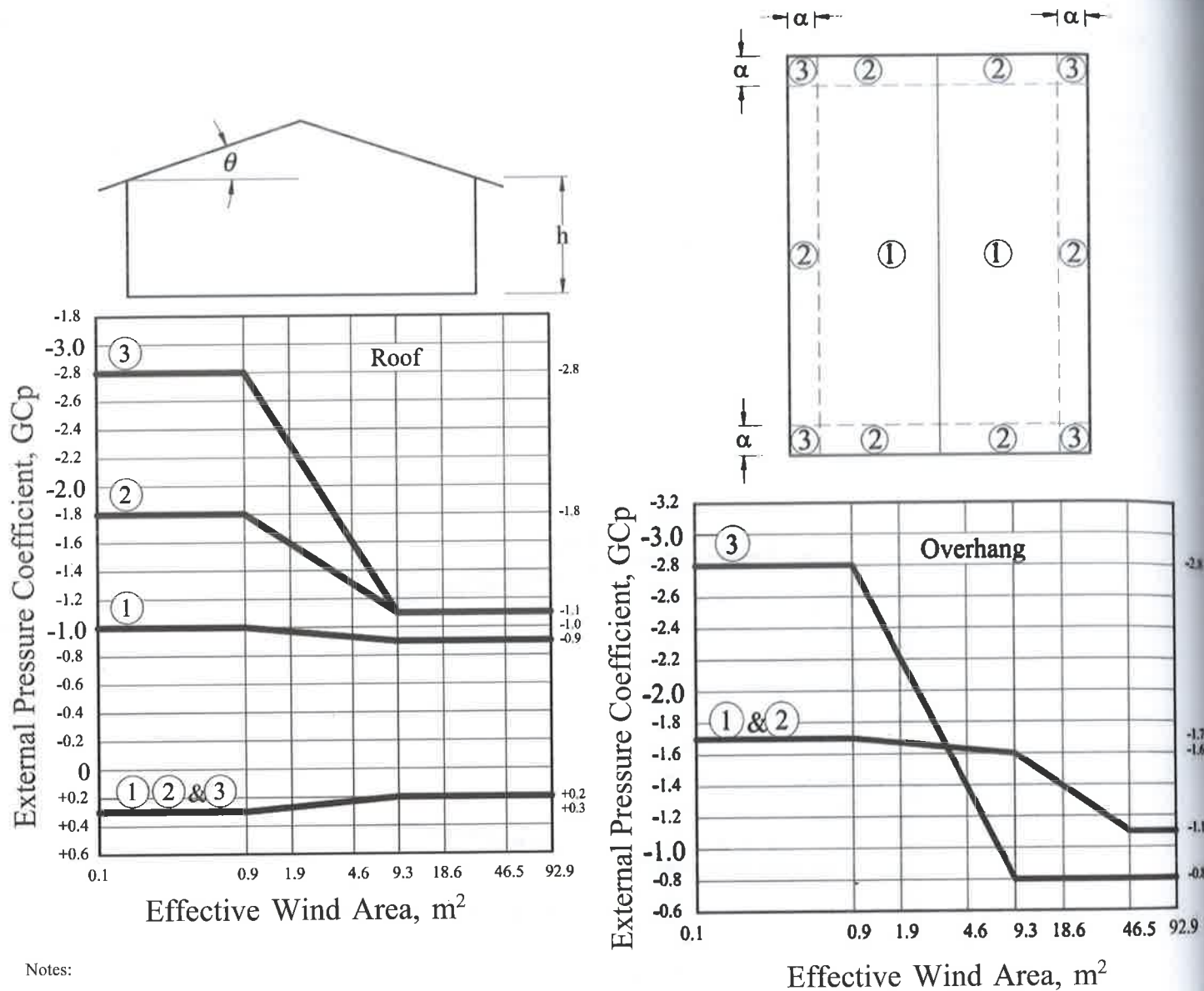
The components and cladding pressure on each wall of the rooftop structure shall be equal to the lateral force determined in accordance with Section 207D.5.1 divided by the respective wall surface area of the rooftop structure and shall be considered to act inward and outward. The components and cladding pressure on the roof shall be equal to the vertical uplift force determined in accordance with Section 207D.5.1 divided by the horizontal projected area of the roof of the rooftop structure and shall be considered to act in the upward direction.



Notes:

1. Vertical scale denotes GC_p to be used with q_h .
2. Horizontal scale denotes effective wind area, m^2 .
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for walls shall be reduced by 10% when $\theta \leq 10^\circ$.
6. Notation:
 - a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
 - h = mean roof height, m, except that eave height shall be used for $\theta \leq 10^\circ$.
 - θ = angle of plane of roof from horizontal, $^\circ$

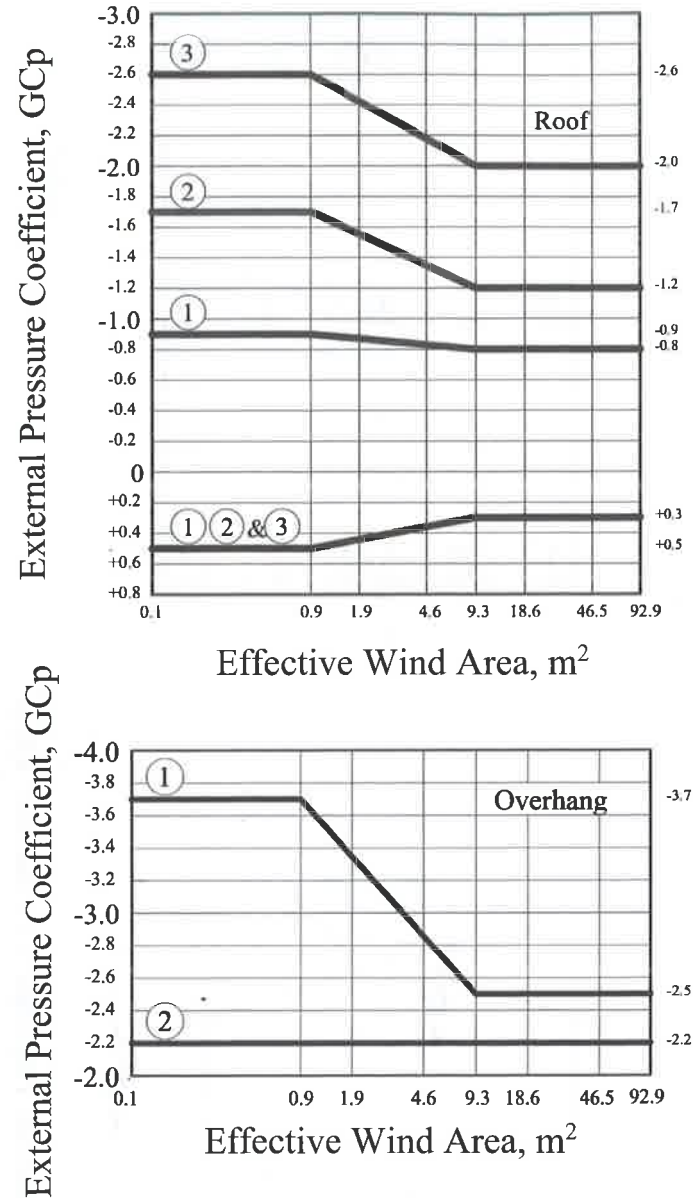
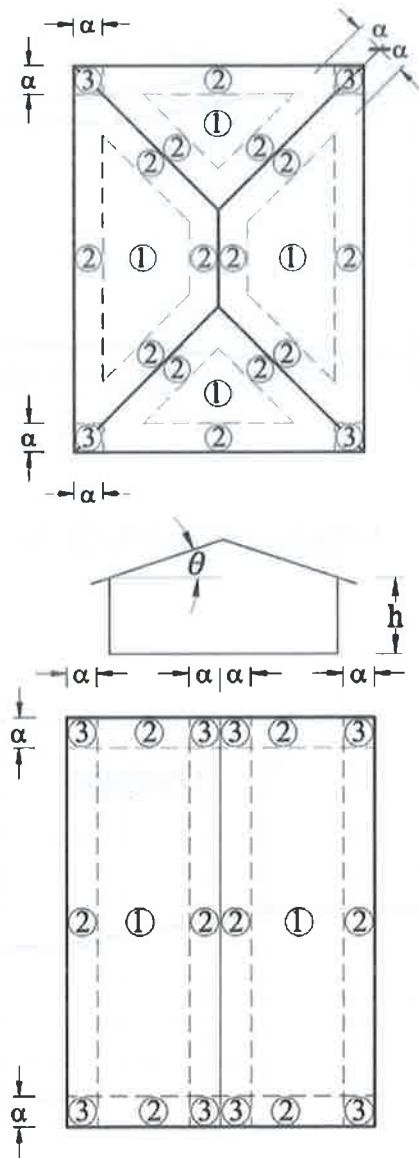
Figure 207E.4-1
External Pressure Coefficients, GC_p Walls, $h \leq 18$ m Enclosed
Partially Enclosed Buildings



Notes:

1. Vertical scale denotes GC_p to be used with q_h .
2. Horizontal scale denotes effective wind area, m^2 .
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. If a parapet equal to or higher than 0.9 m is provided around the perimeter of the roof with $\theta \leq 7^\circ$, the negative values of GC_p in Zone 3 shall be equal to those for Zone 2 and positive values of GC_p in Zones 2 and 3 shall be set equal to those for wall Zones 4 and 5 respectively in Figure 207E.4-1.
6. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
7. Notation:
 - a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
 - h = eave height shall be used for $\theta \leq 10^\circ$.
 - θ = angle of plane of roof from horizontal, $^\circ$

Figure 207E.4-2A
External Pressure Coefficients, GC_p Gable Roofs $\theta \leq 7^\circ$, $h \leq 18$ m
Enclosed, Partially Enclosed Buildings

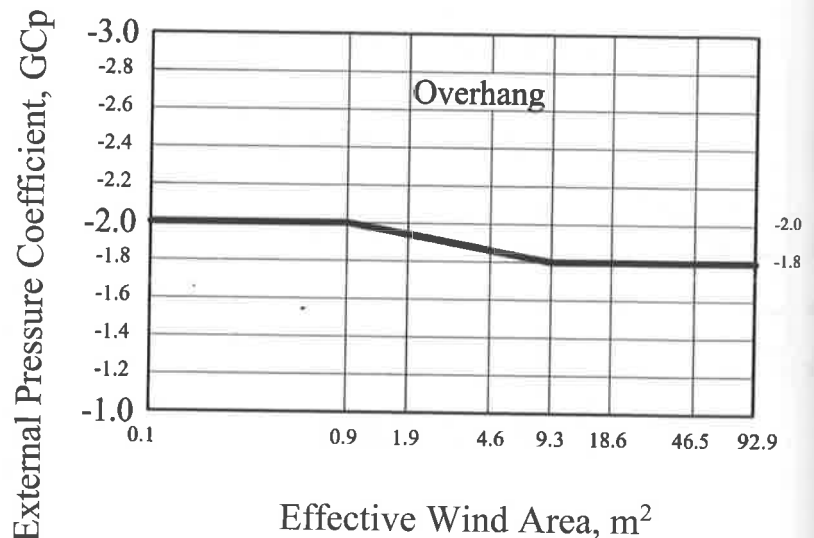
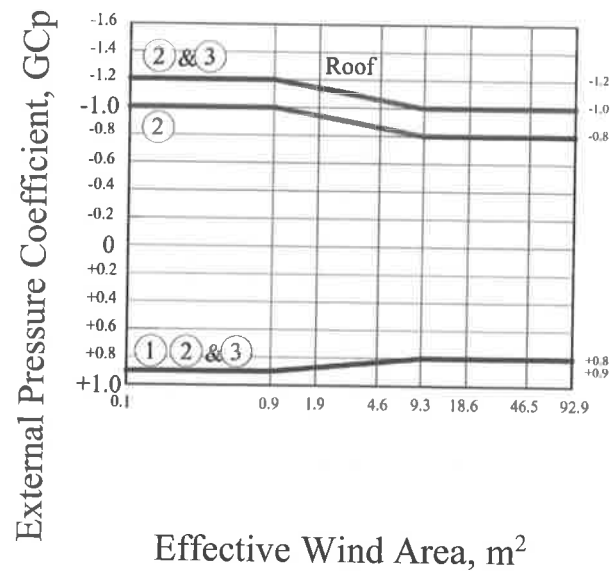
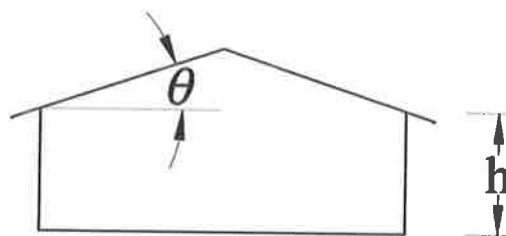
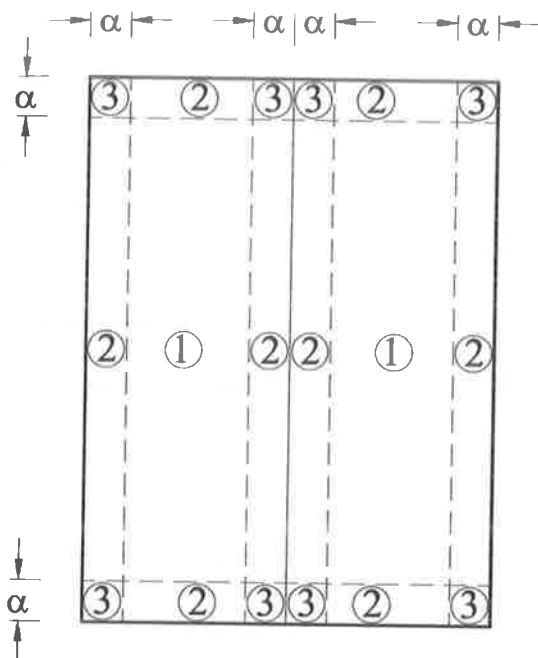


Notes:

1. Vertical scale denotes GC_p to be used with q_h .
2. Horizontal scale denotes effective wind area, m².
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
6. For hip roofs with $7^\circ < \theta \leq 27^\circ$, edge/ridge strips and pressure coefficients for ridges of gabled roofs shall apply on each hip.
7. For hip roofs with $\theta \leq 25^\circ$, Zone 3 shall be treated as Zone 2.
8. Notation:

- A** = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension 0.9 m.
h = mean roof height, m, except that eave height shall be used for $\theta \leq 10^\circ$.
 θ = angle of plane of roof from horizontal, °

Figure 207E.4-2B
 External Pressure Coefficients, GC_p Gable/Hip Roofs $7^\circ < \theta \leq 27^\circ$, $h \leq 18$ m
 Enclosed, Partially Enclosed Buildings



Notes:

1. Vertical scale denotes GC_p to be used with q_h .
2. Horizontal scale denotes effective wind area, m^2 .
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
6. Notation:

- a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
 h = mean roof height, m
 θ = angle of plane of roof from horizontal, °

Figure 207E.4-2C
 External Pressure Coefficients, GC_p , Gable Roofs $27^\circ < \theta \leq 45^\circ$, $h \leq 18$ m
 Enclosed, Partially Enclosed Buildings

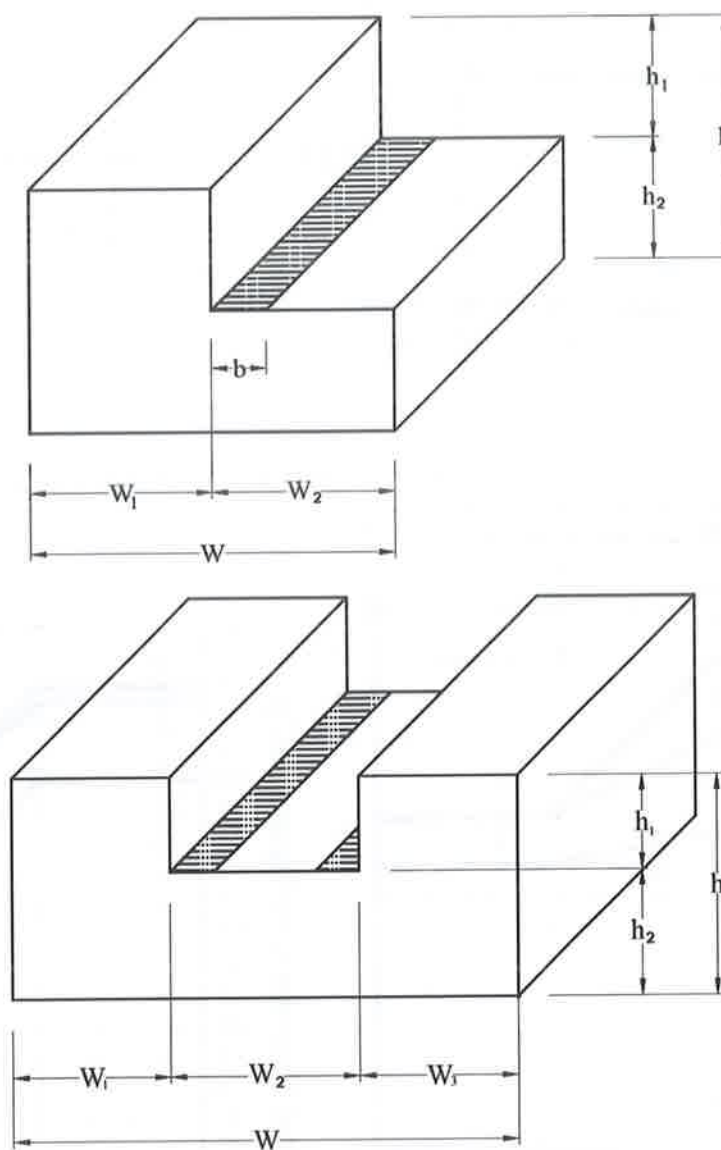
$$h_1 \geq 3 \text{ m}$$

$$b = 1.5h_1$$

$$b < 30 \text{ m}$$

$$\frac{h_1}{h} = 0.30 \text{ to } 0.70$$

$$\frac{w_1}{w} = 0.25 \text{ to } 0.75$$

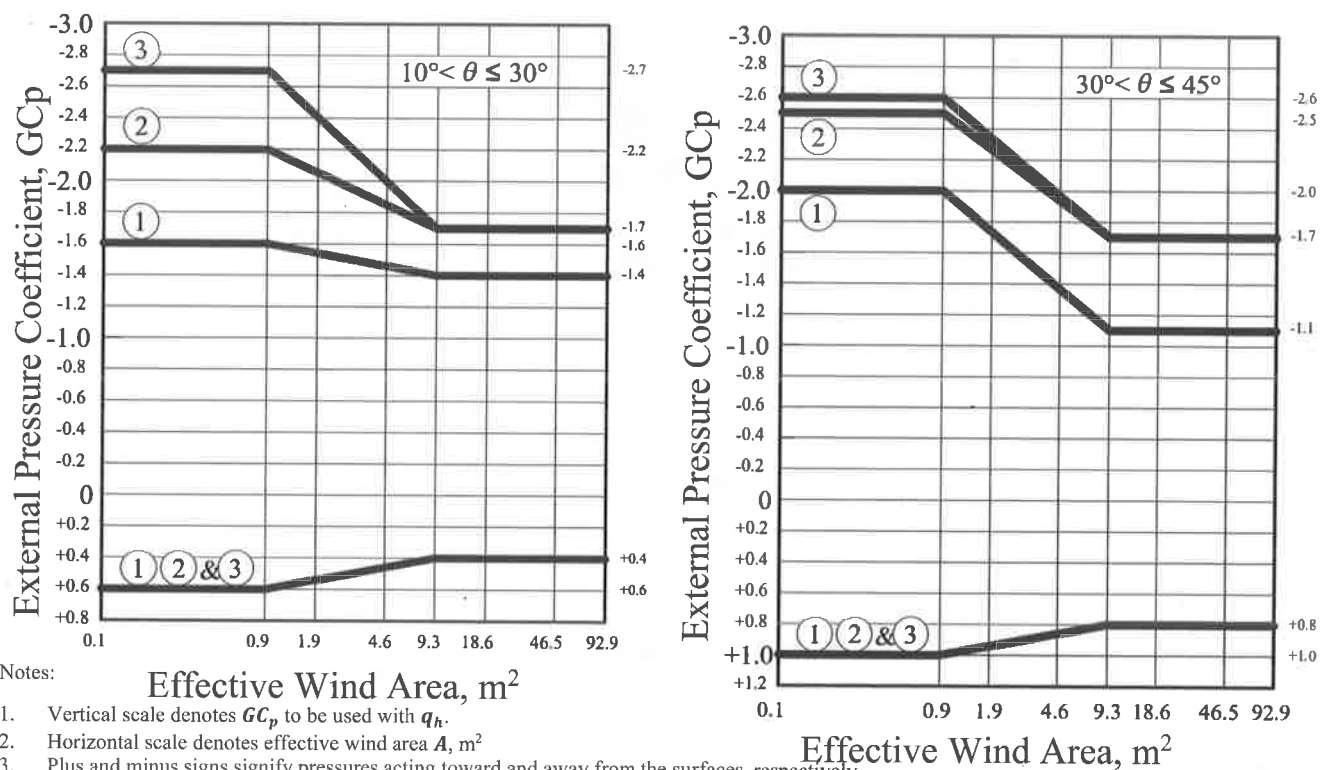
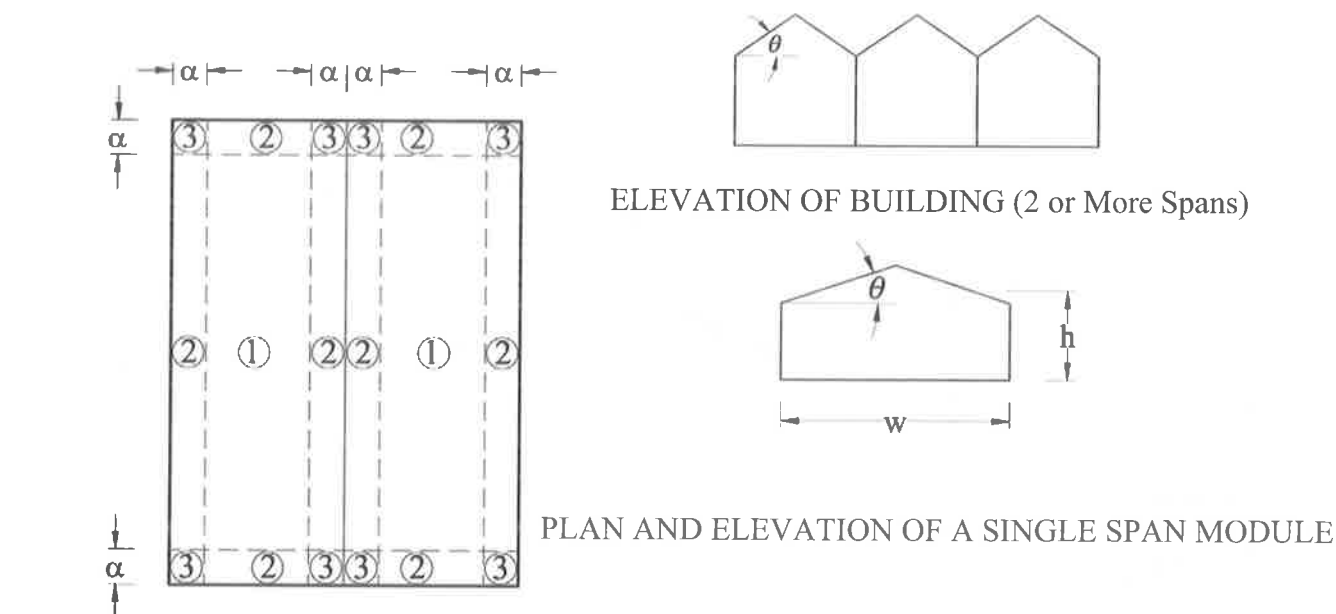


Notes:

1. On the lower level of flat, stepped roofs shown in Figure 207E.4-3, the zone designations and pressure coefficients shown in Figure 207E.4-2A shall apply, except that at the roof-upper wall intersection(s), Zone 3 shall be treated as Zone 2 and Zone 2 shall be treated as Zone 1. Positive values of GC_p equal to those for walls in Figure 207E.4-1 shall apply on the cross-hatched areas shown in Figure 207E.4-3.
2. Notation:

b	=	$1.5h_1$ in Figure 207E.4-3, but not greater than 30 m.
h	=	mean roof height, m
h_i	=	h_1 or h_2 in Figure 207E.4-3; $h = h_1 + h_2$; $h_1 \geq 3.1 \text{ m}$; $h_i/h = 0.3$ to 0.7 .
W	=	Building width in Figure 207E.4-3.
W_i	=	W_1 or W_2 or W_3 in Figure 207E.4-3. $W = W_1 + W_2$ or $W_1 + W_2 + W_3$; $W_i/W = 0.25$ to 0.75 .
θ	=	Angle of plane of roof from horizontal, °

Figure 207E.4-3
External Pressure Coefficients, GC_p Stepped Roofs, $h \leq 18 \text{ m}$
Enclosed, Partially Enclosed Buildings

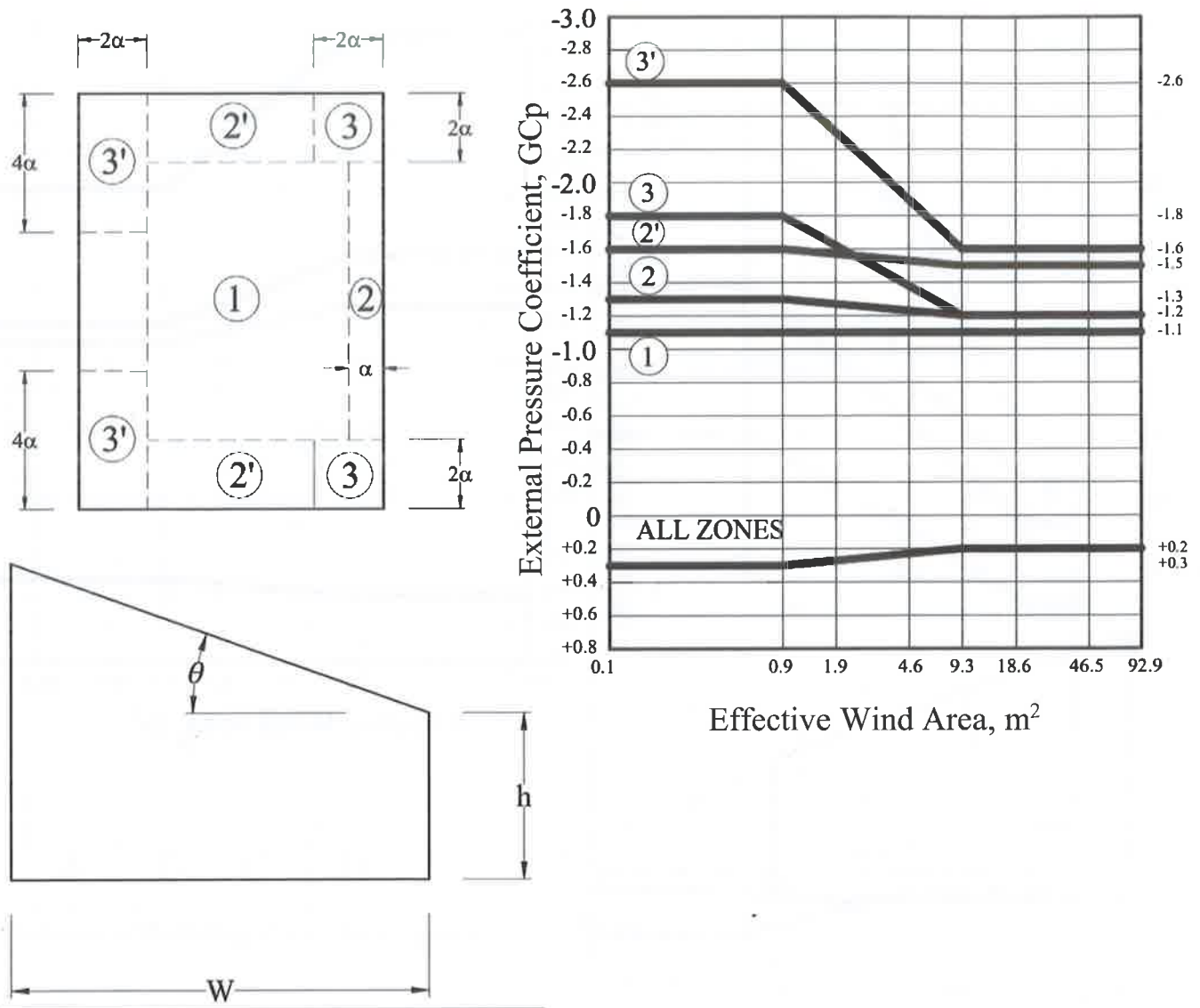


Notes:

1. Vertical scale denotes GC_p to be used with q_h .
2. Horizontal scale denotes effective wind area A , m^2 .
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \leq 10^\circ$, values of GC_p from Figure 207E.4-2A shall be used.
6. Notation:

- a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
 h = mean roof height, m, except that the eave height shall be used for $\theta \leq 10^\circ$
 w = building module width, m
 θ = angle of plane of roof from horizontal, $^\circ$

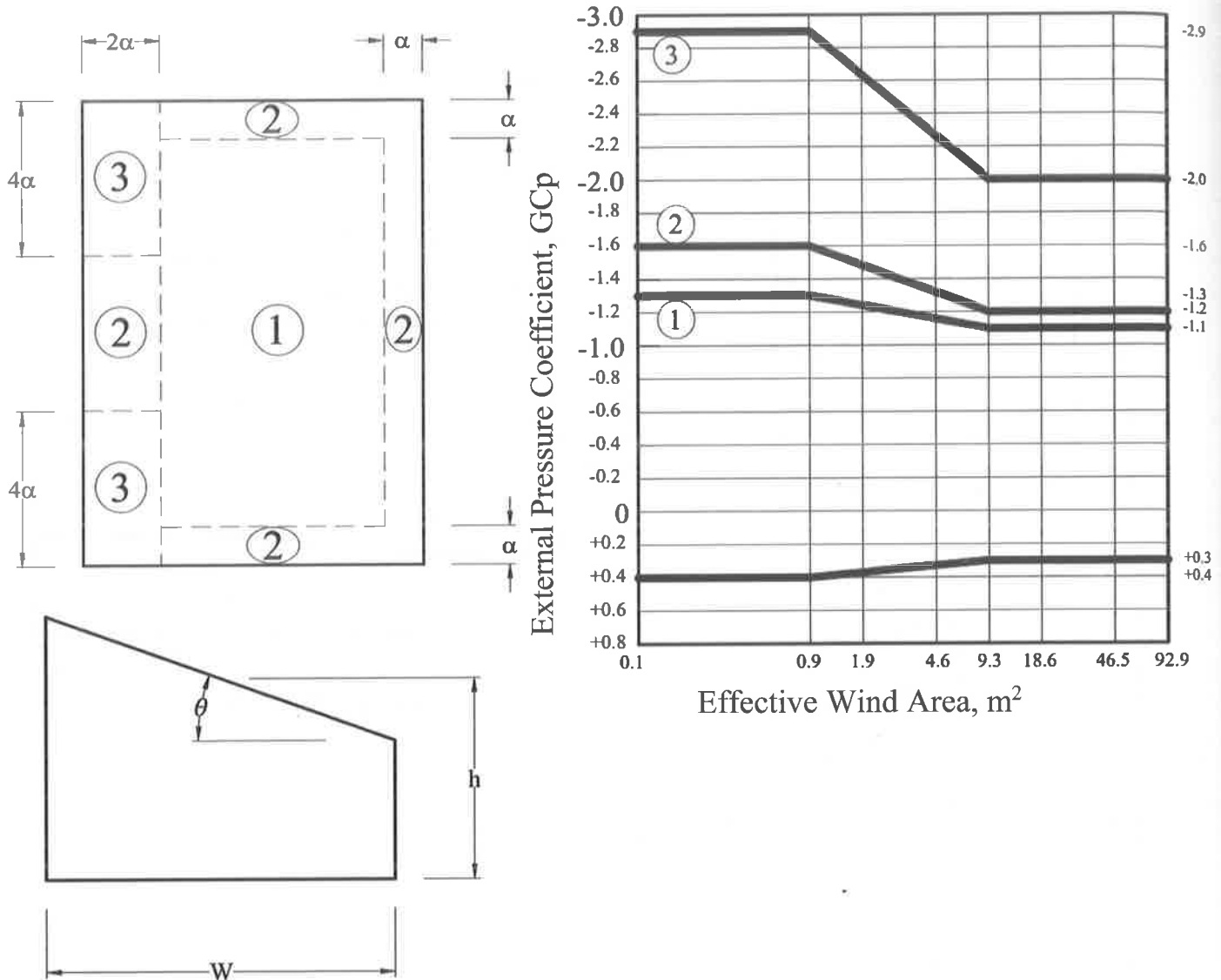
Figure 207E.4-4
External Pressure Coefficients, GC_p , Multispan Gable Roofs, $h \leq 18$ m
Enclosed, Partially Enclosed Buildings



Notes:

1. Vertical scale denotes GC_p to be used with q_h .
2. Horizontal scale denotes effective wind area A , m^2 .
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \leq 3^\circ$, values of GC_p from Figure 207E.4-2A shall be used.
6. Notation:
 - a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
 - h = eave height shall be used for $\theta \leq 10^\circ$
 - W = building width, m
 - θ = angle of plane of roof from horizontal, $^\circ$

Figure 207E.4-5A
External Pressure Coefficients, GC_p Monoslope Roofs, $3^\circ < \theta \leq 10^\circ$, $h \leq 18$ m
Enclosed, Partially Enclosed Buildings

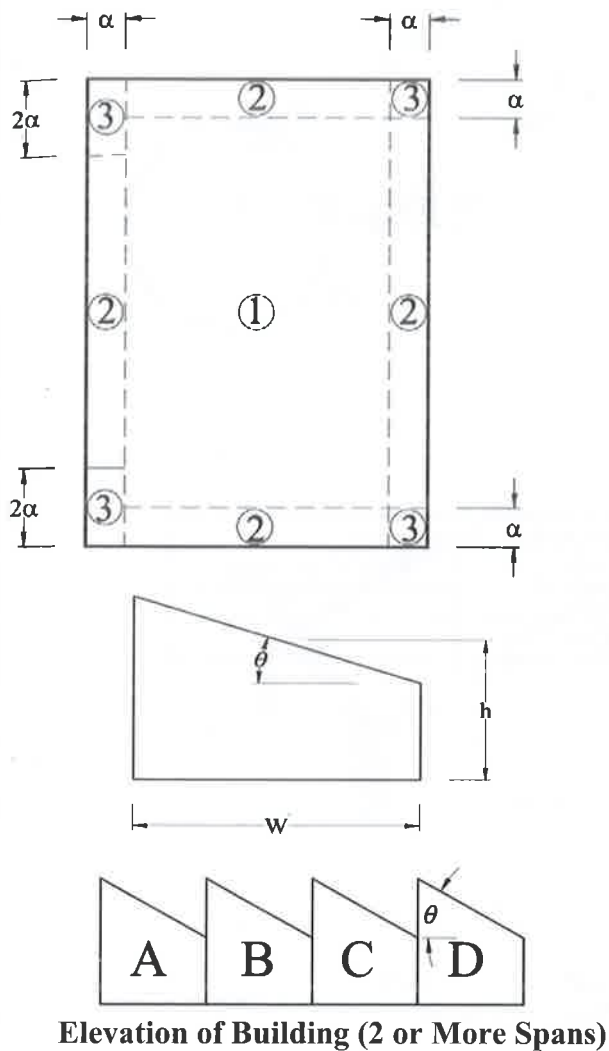


Notes:

1. Vertical scale denotes GC_p to be used with q_h .
2. Horizontal scale denotes effective wind area A , m^2 .
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Notation:

- a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m
 h = mean roof height, m
 W = building width, m
 θ = angle of plane of roof from horizontal, $^\circ$

Figure 207E.4-5B
External Pressure Coefficients, GC_p Monoslope Roofs, $10^\circ < \theta \leq 30^\circ$, $h \leq 18$ m
Enclosed, Partially Enclosed Buildings


Notes:

1. Vertical scale denotes GC_p to be used with q_h .
2. Horizontal scale denotes effective wind area A , m^2
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \leq 10^\circ$, values of GC_p from Figure 207E.4-2A shall be used.
6. Notation:

- a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
 h = mean roof height, m, except that the eave height shall be used for $\theta \leq 10^\circ$
 W = building module width, m
 θ = angle of plane of roof from horizontal, $^\circ$

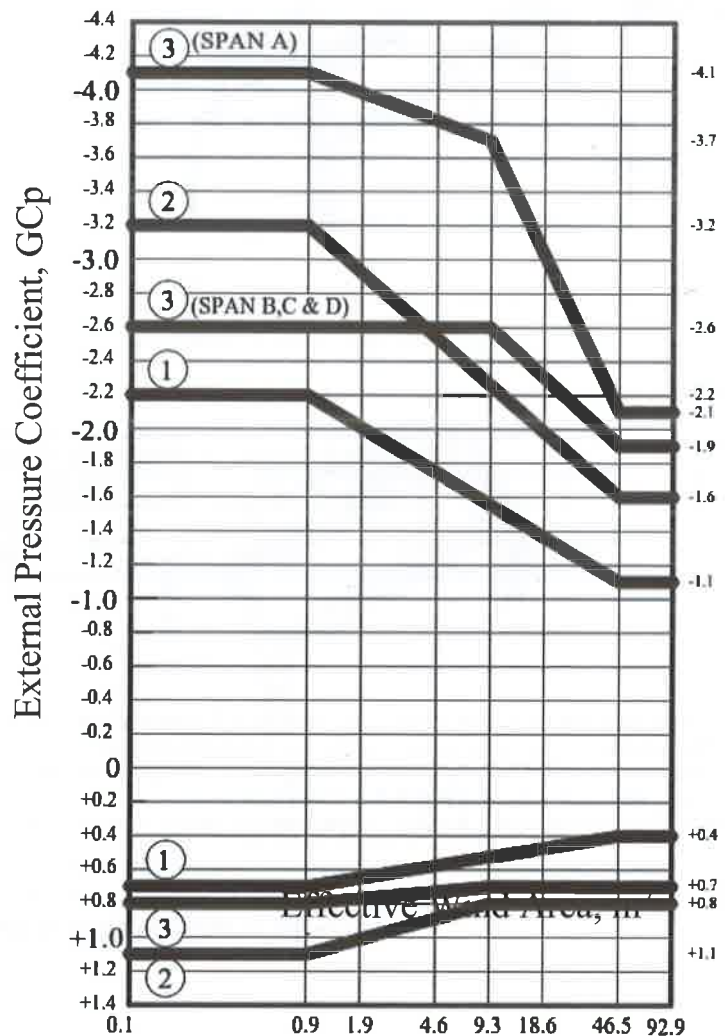
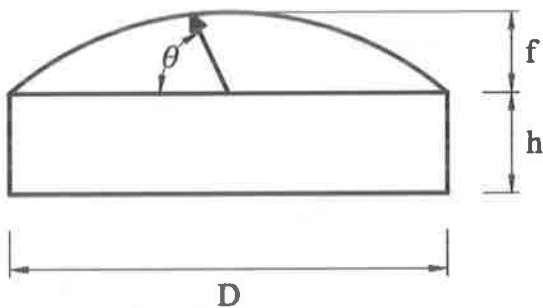
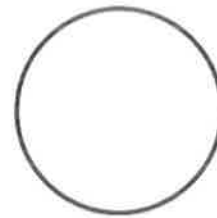


Figure 207E.4-6
 External Pressure Coefficients, GC_p Sawtooth Roofs, $h \leq 18$ m
 Enclosed, Partially Enclosed Buildings

Wind →



Wind →

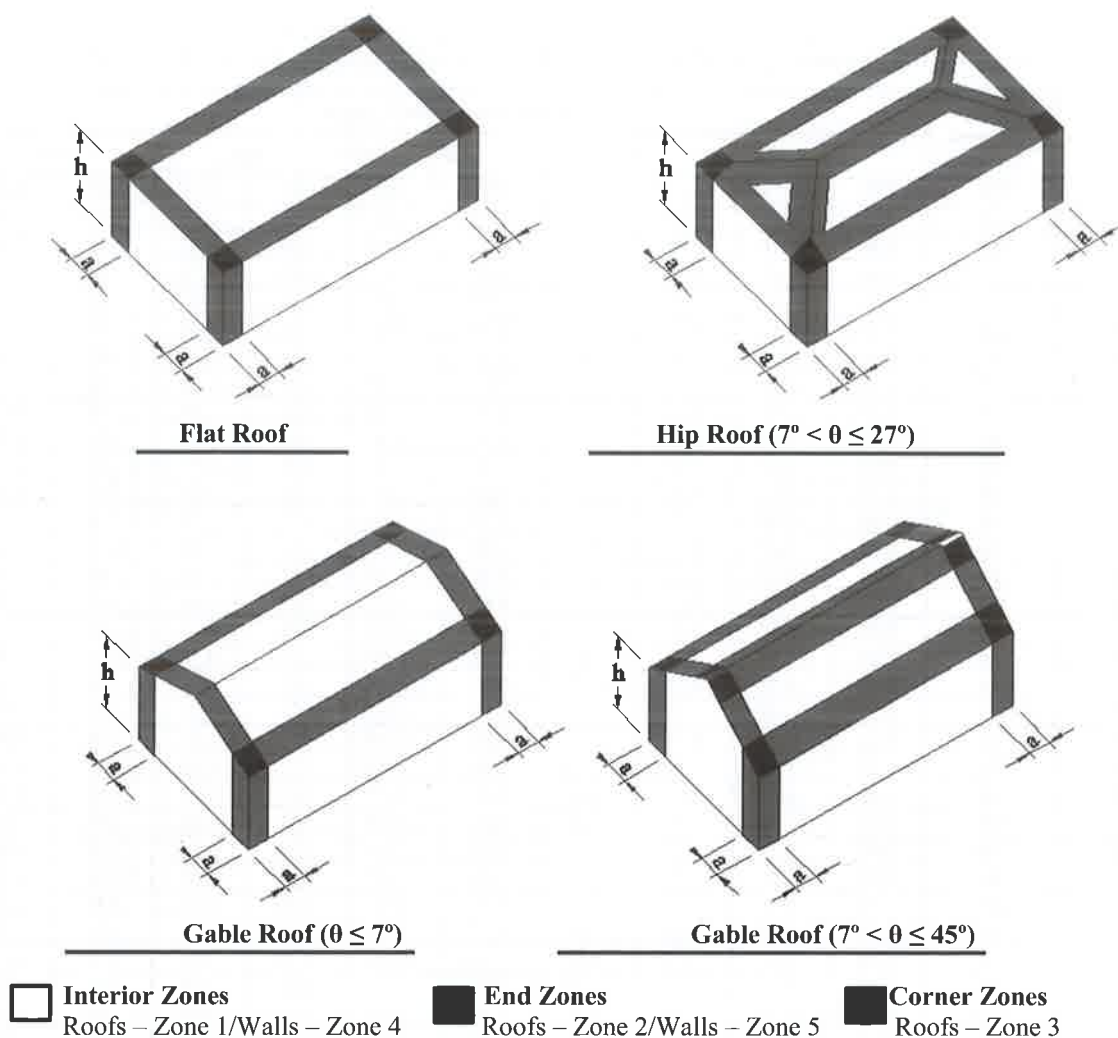


External Pressure Coefficient for Domes with a Circular Base			
θ , degrees	Negative Pressures	Positive Pressures	Positive Pressures
	0 – 90	0 – 60	61 – 90
GC_p	-0.90	+0.90	+0.50

Notes:

1. Values denote GC_p to be used with $q_{(h_D+f)}$, where $h_D + f$ is the height at the top of the dome.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. Each component shall be designed for maximum positive and negative pressures.
4. Values apply to $0 \leq h_D/D \leq 0.5$, $0.2 \leq f/D \leq 0.5$.
5. $\theta = 0^\circ$ on dome springline, $\theta = 90^\circ$ at dome center top point. f is measured from spring line to top.

Figure 207E.4-7
External Pressure Coefficients, GC_p Domed Roofs, All Heights
Enclosed, Partially Enclosed Buildings and Structures



Notes:

1. Pressures shown are applied normal to the surface, for exposure B, at $h = 9$ m. Adjust to other conditions using Equation 207E.5-1.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For hip roofs with $\theta \leq 25^\circ$, Zone 3 shall be treated as Zone 2.
4. For effective wind areas between those given, value may be interpolated, otherwise use the value associated with the lower effective wind area.
5. Notation:

a	=	10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
h	=	mean roof height, m, except that the eave height shall be used for $\theta < 10^\circ$
θ	=	angle of plane of roof from horizontal, $^\circ$

Figure 207E.5-1
Design Wind Pressures Walls and Roofs, $h \leq 18$ m
Enclosed Buildings

Net Design Wind Pressure, p_{net} (kPa) (Exposure B at $h = 10$ m with $I = 1.0$ and $K_d = 1.0$)

	Zone	Effective wind area (sq. m.)	Basic Wind Speed V (kph)									
			150		200		250		300		350	
Roof 0 to 7 degrees	1	1.0	0.30	-0.75	0.54	-1.33	0.84	-2.08	1.22	-2.99	1.65	-4.07
	1	2.0	0.28	-0.73	0.51	-1.29	0.79	-2.02	1.14	-2.91	1.55	-3.96
	1	4.5	0.26	-0.70	0.46	-1.25	0.72	-1.95	1.04	-2.81	1.41	-3.83
	1	9.5	0.24	-0.68	0.43	-1.22	0.67	-1.90	0.96	-2.74	1.31	-3.72
	2	1.0	0.30	-1.25	0.54	-2.23	0.84	-3.48	1.22	-5.02	1.65	-6.83
	2	2.0	0.28	-1.12	0.51	-1.99	0.79	-3.11	1.14	-4.48	1.55	-6.10
	2	4.5	0.26	-0.94	0.46	-1.68	0.72	-2.62	1.04	-3.77	1.41	-5.14
	2	9.5	0.24	-0.81	0.43	-1.44	0.67	-2.25	0.96	-3.24	1.31	-4.41
	3	1.0	0.30	-1.89	0.54	-3.36	0.84	-5.25	1.22	-7.56	1.65	-10.29
	3	2.0	0.28	-1.56	0.51	-2.78	0.79	-4.34	1.14	-6.26	1.55	-8.52
	3	4.5	0.26	-1.14	0.46	-2.02	0.72	-3.16	1.04	-4.55	1.41	-6.19
	3	9.5	0.24	-0.81	0.43	-1.44	0.67	-2.25	0.96	-3.24	1.31	-4.41
Roof > 7 to 27 degrees	1	1.0	0.43	-0.68	0.77	-1.22	1.20	-1.90	1.72	-2.74	2.34	-3.72
	1	2.0	0.39	-0.66	0.70	-1.18	1.09	-1.85	1.57	-2.66	2.14	-3.62
	1	4.5	0.34	-0.64	0.61	-1.14	0.95	-1.78	1.37	-2.56	1.86	-3.48
	1	9.5	0.31	-0.62	0.55	-1.10	0.85	-1.72	1.23	-2.48	1.67	-3.38
	2	1.0	0.43	-1.19	0.77	-2.12	1.20	-3.32	1.72	-4.77	2.34	-6.50
	2	2.0	0.39	-1.10	0.70	-1.95	1.09	-3.04	1.57	-4.38	2.14	-5.96
	2	4.5	0.34	-0.97	0.61	-1.72	0.95	-2.69	1.37	-3.88	1.86	-5.27
	2	9.5	0.31	-0.88	0.55	-1.56	0.85	-2.44	1.23	-3.51	1.67	-4.77
	3	1.0	0.43	-1.76	0.77	-3.14	1.20	-4.90	1.72	-7.05	2.34	-9.60
	3	2.0	0.39	-1.65	0.70	-2.93	1.09	-4.57	1.57	-6.59	2.14	-8.96
	3	4.5	0.34	-1.49	0.61	-2.66	0.95	-4.15	1.37	-5.98	1.86	-8.14
	3	9.5	0.31	-1.38	0.55	-2.45	0.85	-3.83	1.23	-5.52	1.67	-7.52
Roof > 27 to 45 degrees	1	1.0	0.68	-0.75	1.22	-1.33	1.90	-2.08	2.74	-3.00	3.72	-4.09
	1	2.0	0.66	-0.71	1.18	-1.27	1.85	-1.98	2.66	-2.85	3.62	-3.88
	1	4.5	0.64	-0.66	1.14	-1.17	1.78	-1.83	2.56	-2.63	3.48	-3.59
	1	9.5	0.62	-0.62	1.10	-1.10	1.72	-1.72	2.48	-2.48	3.38	-3.38
	2	1.0	0.68	-0.88	1.22	-1.56	1.90	-2.44	2.74	-3.51	3.72	-4.77
	2	2.0	0.66	-0.84	1.18	-1.49	1.85	-2.32	2.66	-3.34	3.62	-4.55
	2	4.5	0.64	-0.79	1.14	-1.40	1.78	-2.18	2.56	-3.14	3.48	-4.27
	2	9.5	0.62	-0.75	1.10	-1.33	1.72	-2.08	2.48	-3.00	3.38	-4.09
	3	1.0	0.68	-0.88	1.22	-1.56	1.90	-2.44	2.74	-3.51	3.72	-4.77
	3	2.0	0.66	-0.84	1.18	-1.49	1.85	-2.32	2.66	-3.34	3.62	-4.55
	3	4.5	0.64	-0.79	1.14	-1.40	1.78	-2.18	2.56	-3.14	3.48	-4.27
	3	9.5	0.62	-0.75	1.10	-1.33	1.72	-2.08	2.48	-3.00	3.38	-4.09
Wall	4	1.0	0.75	-0.81	1.33	-1.44	2.08	-2.25	2.99	-3.24	4.07	-4.41
	4	2.0	0.72	-0.78	1.27	-1.38	1.99	-2.16	2.86	-3.12	3.90	-4.24
	4	4.5	0.67	-0.73	1.19	-1.30	1.86	-2.03	2.68	-2.93	3.65	-3.98
	4	9.5	0.64	-0.70	1.13	-1.24	1.77	-1.94	2.55	-2.80	3.46	-3.81
	4	46.5	0.56	-0.62	0.99	-1.10	1.55	-1.72	2.23	-2.48	3.03	-3.38
	5	1.0	0.75	-1.00	1.33	-1.78	2.08	-2.78	2.99	-4.00	4.07	-5.45
	5	2.0	0.72	-0.94	1.27	-1.67	1.99	-2.60	2.86	-3.75	3.90	-5.10
	5	4.5	0.67	-0.85	1.19	-1.50	1.86	-2.35	2.68	-3.38	3.65	-4.60
	5	9.5	0.64	-0.78	1.13	-1.38	1.77	-2.15	2.55	-3.10	3.46	-4.22
	5	46.5	0.56	-0.62	0.99	-1.10	1.55	-1.72	2.23	-2.48	3.03	-3.38

Note: For effective areas between those given above the load may be interpolated, otherwise use the load associated with the lower effective area. The final value, including all permitted reductions, used in the design shall not be less than that required by Section 207E.2.2.

Figure 207E.5-1 (continued)
Design Wind Pressures on Walls and Roofs of Enclosed Buildings with $h \leq 18$ m
Components and Cladding

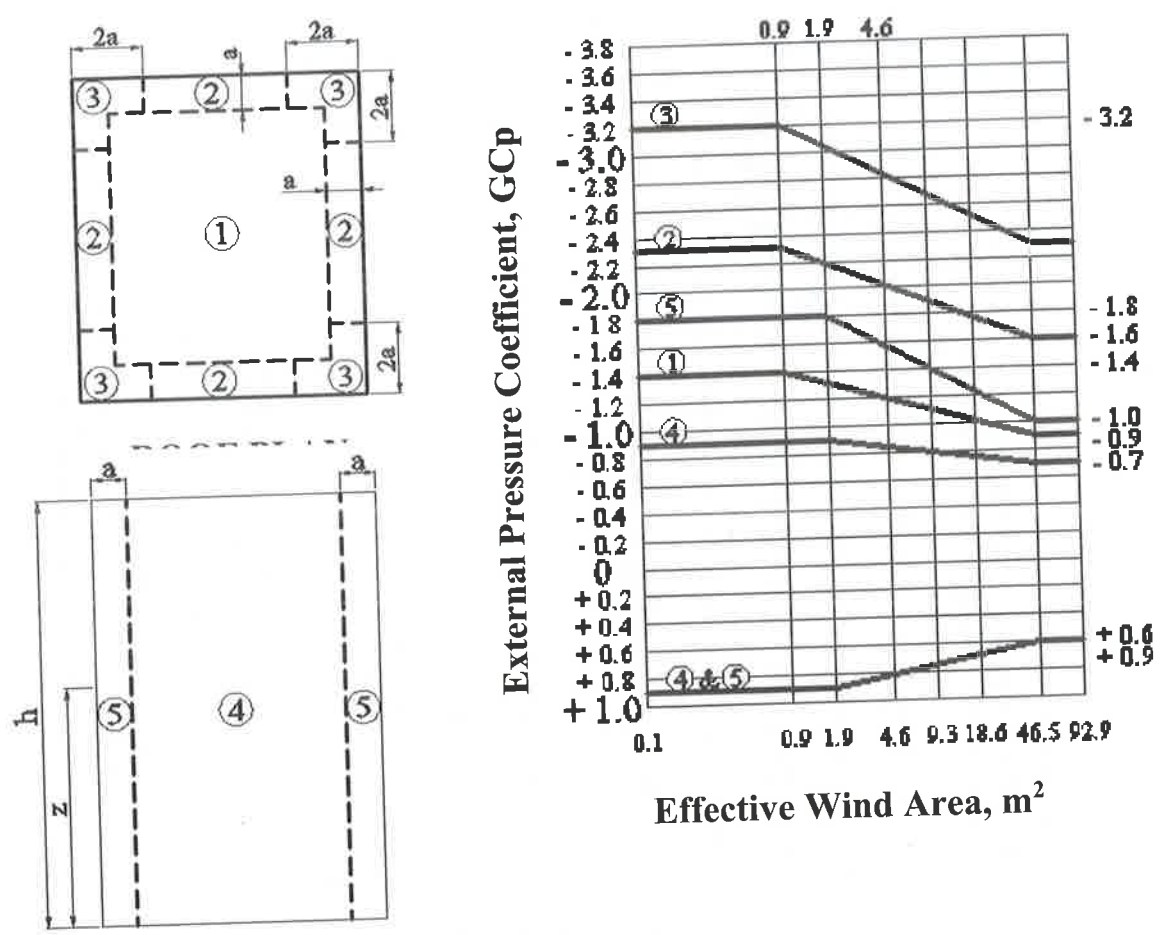
Roof Overhang Net Design Wind Pressure, p_{net} (kPa)
(Exposure B at $h = 10$ m with $I = 1.0$ and $K_d = 1.0$)

	Zone	Effective wind area (sq.m)	Basic Wind Speed V (kph)				
			150	200	250	300	350
Roof 0 to 7 degrees	2	0.9	-1.19	-2.12	-3.31	-4.76	-6.48
	2	1.9	-1.17	-2.08	-3.25	-4.69	-10.27
	2	4.6	-1.14	-2.03	-3.17	-4.57	-6.38
	2	9.3	-1.13	-2.00	-3.13	-4.51	-8.21
	3	0.9	-1.89	-3.35	-5.24	-7.55	-6.22
	3	1.9	-1.51	-2.68	-4.19	-6.03	-5.45
	3	4.6	-1.00	-1.78	-2.78	-4.00	-6.14
	3	9.3	-1.25	-2.23	-3.48	-5.02	-6.83
Roof > 7 to 27 degrees	2	0.9	-1.51	-2.68	-4.19	-6.03	-8.21
	2	1.9	-1.51	-2.68	-4.19	-6.03	-13.38
	2	4.6	-1.51	-2.68	-4.19	-6.03	-8.21
	2	9.3	-1.51	-2.68	-4.19	-6.03	-12.17
	3	0.9	-2.46	-4.37	-6.82	-9.83	-8.21
	3	1.9	-2.24	-3.97	-6.21	-8.94	-10.45
	3	4.6	-1.92	-3.41	-5.33	-7.67	-8.21
	3	9.3	-1.70	-3.02	-4.71	-6.79	-9.24
Roof > 27 to 45 degrees	2	0.9	-1.38	-2.45	-3.83	-5.52	-7.52
	2	1.9	-1.35	-2.40	-3.75	-5.40	-7.52
	2	4.6	-1.29	-2.29	-3.57	-5.14	-7.34
	2	9.3	-1.25	-2.23	-3.48	-5.02	-7.34
	3	0.9	-1.38	-2.45	-3.83	-5.52	-7.00
	3	1.9	-1.35	-2.40	-3.75	-5.40	-7.00
	3	4.6	-1.29	-2.29	-3.57	-5.14	-6.83
	3	9.3	-1.25	-2.23	-3.48	-5.02	-6.83

**Adjustment Factor
for Building Height and Exposure, λ**

Mean roof height (m)	Exposure		
	B	C	D
4.5	1.00	1.21	1.47
6.0	1.00	1.29	1.55
7.5	1.00	1.35	1.61
9.0	1.00	1.40	1.66
10.5	1.05	1.45	1.70
12.0	1.09	1.49	1.74
13.5	1.12	1.53	1.78
15.0	1.16	1.56	1.81
16.5	1.19	1.59	1.84
18.0	1.22	1.62	1.87

Figure 207E.5-1 (continued)
Design Wind Pressures on Walls and Roofs of Enclosed Buildings with $h \leq 18$ m
Components and Cladding

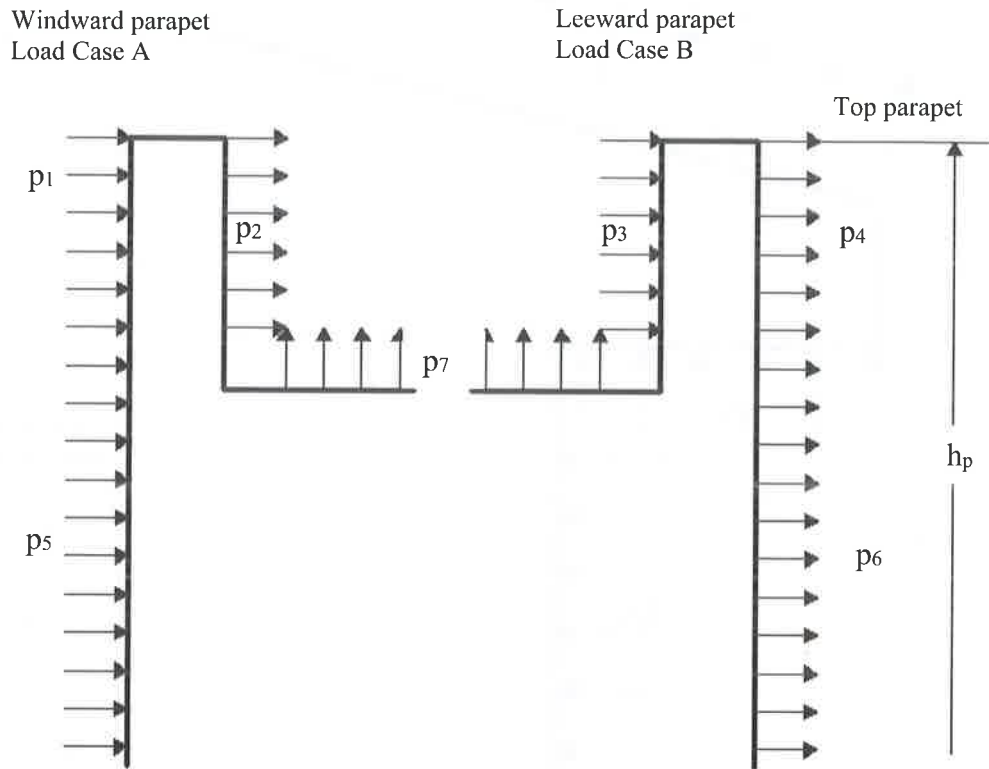


WALL ELEVATION

Notes:

1. Vertical scale denotes GC_p to be used with appropriate q_z or q_h .
2. Horizontal scale denotes effective wind area, m^2 .
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Use q_z with positive values of GC_p and q_h with negative values of GC_p .
5. Each component shall be designed for maximum positive and negative pressures.
6. Coefficients are for roofs with angle $\theta \leq 10^\circ$. For other roof angles and geometry, use GC_p values from Figure 207E.4-2A, B and C and attendant q_h based on exposure defined in Section 207A.7.
7. If a parapet equal to or higher than 0.9 m is provided around the perimeter of the roof with $\theta \leq 10^\circ$, Zone 3 shall be treated as Zone 2.
8. Notation:
 - a = 10% of least horizontal dimension, but not less than 0.9 m.
 - h = mean roof height, m, except that eave height shall be used for $\theta \leq 10^\circ$.
 - z = height above ground, m
 - θ = angle of plane of roof from horizontal, $^\circ$

Figure 207E.6-1
External Pressure Coefficients, GC_p Walls and Roofs, $h > 18$ m
Enclosed, Partially Enclosed Buildings



Windward Parapet

Load Case A

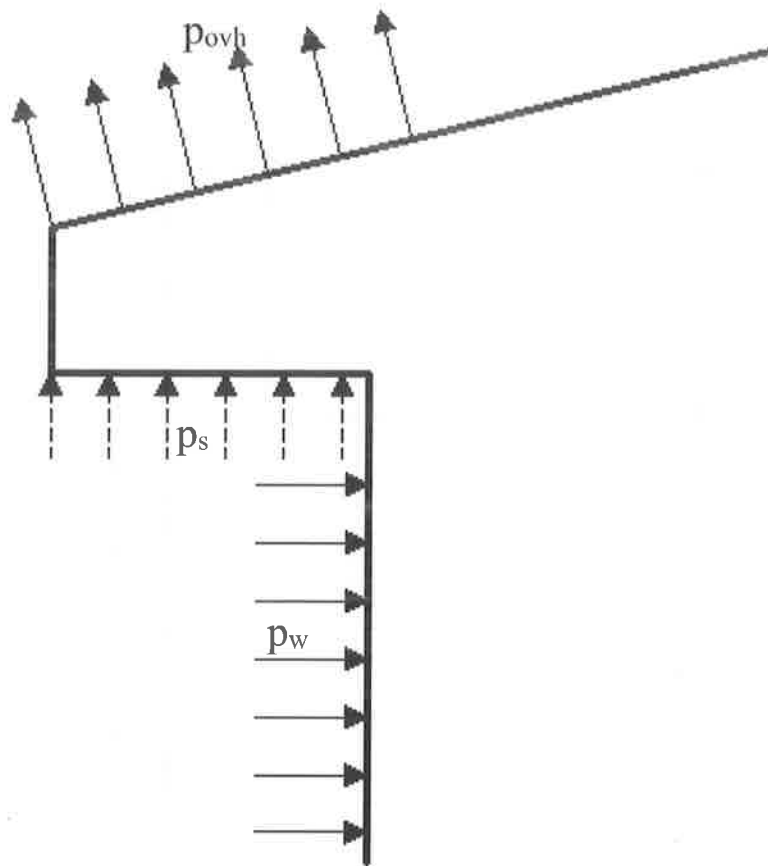
1. Windward parapet pressure (p_1) is determined using the positive wall pressure (p_5) zones 4 or 5 from Table 207E.7-2.
2. Leeward parapet pressure (p_2) is determined using the negative roof pressure (p_7) zones 2 or 3 from Table 207E.7-2.

Leeward Parapet

Load Case B

1. Windward parapet pressure (p_3) is determined using the positive wall pressure (p_5) zones 4 or 5 from Table 207E.7-2.
2. Leeward parapet pressure (p_4) is determined using the negative wall pressure (p_6) zones 4 or 5 from Table 207E.7-2.

Figure 207E.7-1
Parapet Wind Loads Application of Parapet Wind Loads, $h \leq 48.8$ m
Enclosed Simple Diaphragm Buildings



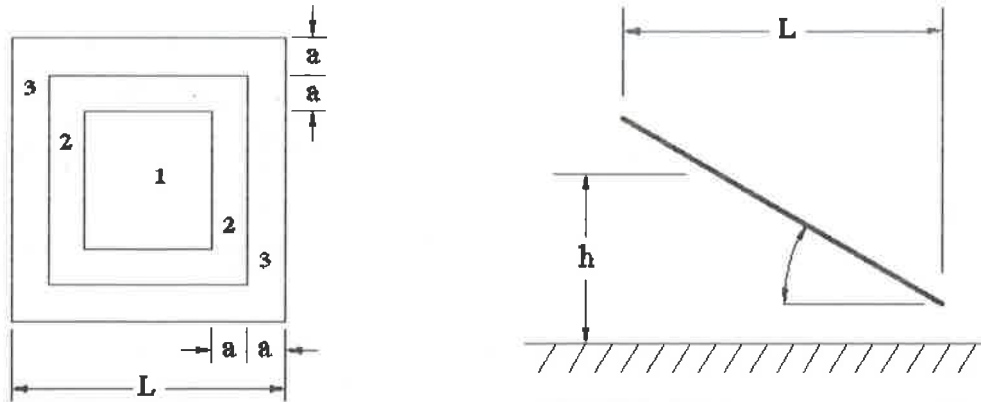
$$p_{ovh} = 1.0 \times \text{roof pressure } p \text{ from tables for edge Zones 1, 2}$$

$$p_{ovh} = 1.15 \times \text{roof pressure } p \text{ from tables for corner Zone 3}$$

Notes:

1. p_{ovh} = roof pressure at overhang for edge or corner zone as applicable from figures in roof pressure table.
2. p_{ovh} from figures includes load from both top and bottom surface of overhang.
3. Pressure p_s at soffit of overhang can be assumed same as wall pressure, p_w .

Figure 207E.7-2
Roof Overhang Wind Loads Application of Overhang Wind Loads, $h \leq 48$ m
Enclosed Simple Diaphragm Buildings



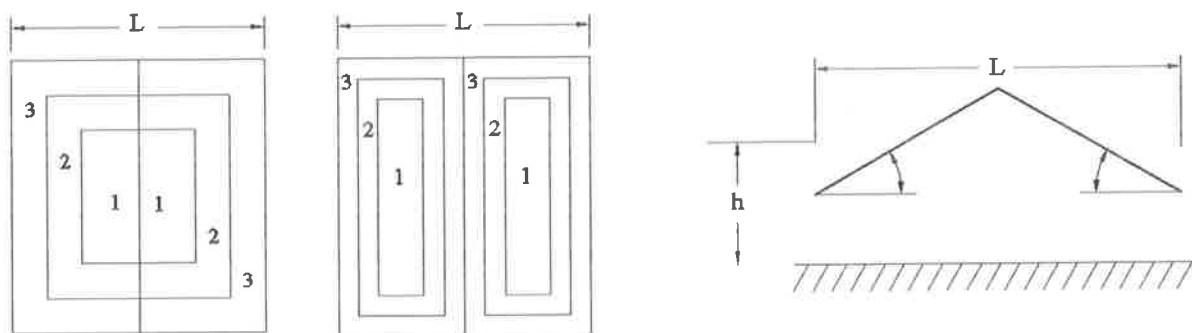
Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	$> a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	$\leq a^2$	3.2	-4.2	2.4	-2.1	1.6	-1.4	1.6	-5.1	1.2	-2.6	0.8	-1.7
	$> a^2, \leq 4.0a^2$	2.4	-2.1	2.4	-2.1	1.6	-1.4	1.2	-2.6	1.2	-2.6	0.8	-1.7
	$> 4.0a^2$	1.6	-1.4	1.6	-1.4	1.6	-1.4	0.8	-1.7	0.8	-1.7	0.8	-1.7
15°	$\leq a^2$	3.6	-3.8	2.7	-2.9	1.8	-1.9	2.4	-4.2	1.8	-3.2	1.2	-2.1
	$> a^2, \leq 4.0a^2$	2.7	-2.9	2.7	-2.9	1.8	-1.9	1.8	-3.2	1.8	-3.2	1.2	-2.1
	$> 4.0a^2$	1.8	-1.9	1.8	-1.9	1.8	-1.9	1.2	-2.1	1.2	-2.1	1.2	-2.1
30°	$\leq a^2$	5.2	-5	3.9	-3.8	2.6	2.5	3.2	-4.6	2.4	-3.5	1.6	-2.3
	$> a^2, \leq 4.0a^2$	3.9	-3.8	3.9	-3.8	2.6	2.5	2.4	-3.5	2.4	-3.5	1.6	-2.3
	$> 4.0a^2$	2.6	-2.5	2.6	-2.5	2.6	2.5	1.6	-2.3	1.6	-2.3	1.6	-2.3
45°	$\leq a^2$	5.2	-4.6	3.9	-3.5	2.6	2.3	4.2	-3.8	3.2	-2.9	2.1	-1.9
	$> a^2, \leq 4.0a^2$	3.9	-3.5	3.9	-3.5	2.6	2.3	3.2	-2.9	3.2	-2.9	2.1	-1.9
	$> 4.0a^2$	2.6	-2.3	2.6	-2.3	2.6	2.3	2.1	-1.9	2.1	-1.9	2.1	-1.9

Notes:

- C_N denotes net pressures (contributions from top and bottom surfaces).
- Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
- For values of θ other than those shown, linear interpolation is permitted.
- Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
- Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
- Notation:

- a = 10% of least horizontal dimension or $0.4h$, whichever is smaller but not less than 4% of least horizontal dimension or 0.9 m.
 h = mean roof height, m
 L = Horizontal dimension of building, measured in along wind direction, m
 θ = angle of plane of roof from horizontal, $^\circ$

Figure 207E.8-1
Net Pressure Coefficient, C_N Monoslope Free Roofs, $\theta \leq 45^\circ$, $0.25 \leq h/L \leq 1.0$
Open Buildings



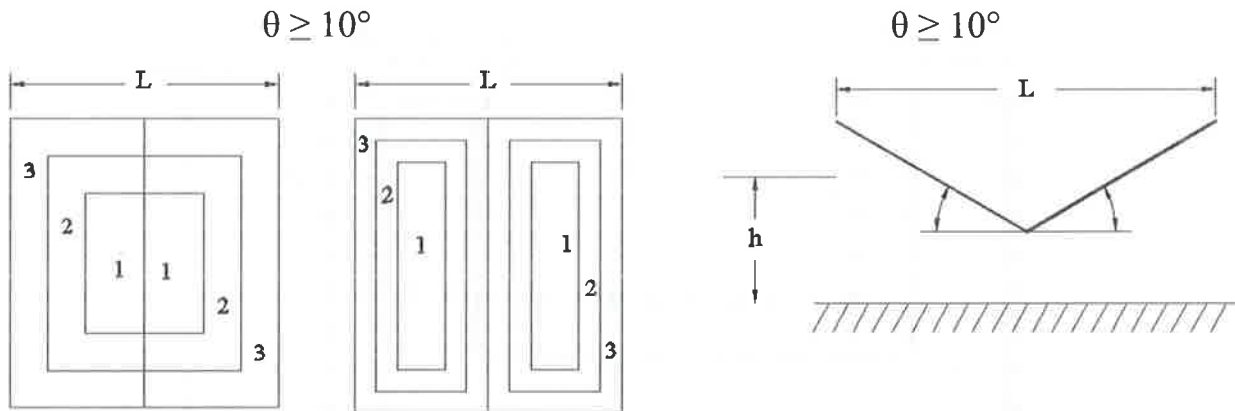
Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	$> a^2,$ $\leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	$\leq a^2$	2.2	-3.6	1.7	-1.8	1.1	-1.2	1	-5.1	0.8	-2.6	0.5	-1.7
	$> a^2,$ $\leq 4.0a^2$	1.7	-1.8	1.7	-1.8	1.1	-1.2	0.8	-2.6	0.8	-2.6	0.5	-1.7
	$> 4.0a^2$	1.1	-1.2	1.2	-1.2	1.1	-1.2	0.5	-1.7	0.5	-1.7	0.5	-1.7
15°	$\leq a^2$	2.2	-2.2	1.7	-1.7	1.1	-1.1	1	-3.2	0.8	-2.4	0.5	-1.6
	$> a^2,$ $\leq 4.0a^2$	1.7	-1.7	1.7	-1.7	1.1	-1.1	0.8	-2.4	0.8	-2.4	0.5	-1.6
	$> 4.0a^2$	1.1	-1.1	1.1	-1.1	1.1	-1.1	0.5	-1.6	0.5	-1.6	0.5	-1.6
30°	$\leq a^2$	2.6	-1.8	2	-1.4	1.3	-0.9	1	-2.4	0.8	-1.8	0.5	-1.2
	$> a^2,$ $\leq 4.0a^2$	2	-1.4	2	-1.4	1.3	-0.9	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	1.3	-0.9	1.3	-0.9	1.3	-0.9	0.5	-1.2	0.5	-1.2	0.5	-1.2
45°	$\leq a^2$	2.2	-1.6	1.7	-1.2	1.1	-0.8	1	-2.4	0.8	-1.8	0.5	-1.2
	$> a^2,$ $\leq 4.0a^2$	1.7	-1.2	1.7	-1.2	1.1	-0.8	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	1.1	-0.8	1.1	-0.8	1.1	-0.8	0.5	-1.2	0.5	-1.2	0.5	-1.2

Notes:

- C_N denotes net pressures (contributions from top and bottom surfaces).
- Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
- For values of θ other than those shown, linear interpolation is permitted.
- Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
- Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
- Notation:

- a = 10% of least horizontal dimension or $0.4h$, whichever is smaller but not less than 4% of least horizontal dimension or 0.9 m. Dimension "a" is as shown in Figure 207E.8-1.
- h = mean roof height, m
- L = Horizontal dimension of building, measured in along wind direction, m
- θ = angle of plane of roof from horizontal, °

Figure 207E.8-2
Net Pressure Coefficient, C_N Pitched Free Roofs, $\theta \leq 45^\circ$, $0.25 \leq h/L \leq 1.0$
Open Buildings

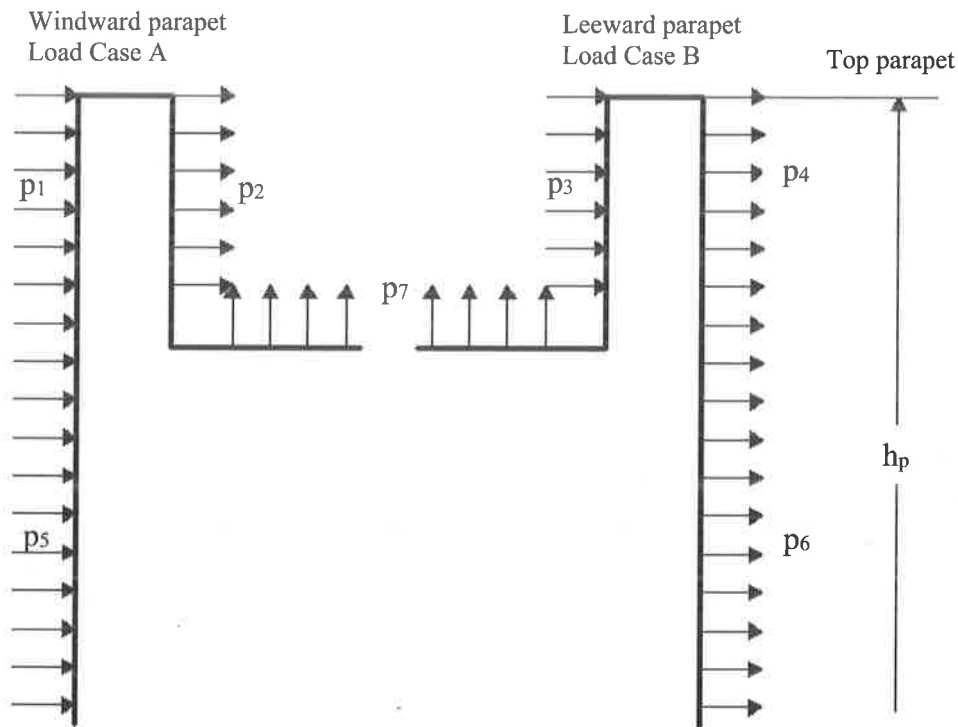


Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	$> a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-4.8	0.8	-2.4	0.5	-1.6
	$> a^2, \leq 4.0a^2$	1.8	-1.1	1.8	-1.7	1.2	-1.1	0.8	-2.4	0.8	-2.4	0.5	-1.6
	$> 4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.6	0.5	-1.6	0.5	-1.6
15°	$\leq a^2$	2.2	-2.2	1.7	-1.7	1.1	-1.1	1	-2.4	0.8	-1.8	0.5	-1.2
	$> a^2, \leq 4.0a^2$	1.7	-1.7	1.7	-1.7	1.1	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	1.1	-1.1	1.1	-1.1	1.1	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
30°	$\leq a^2$	1.8	-2.6	1.4	-2	0.9	-1.3	1	-2.8	0.8	-2.1	0.5	-1.4
	$> a^2, \leq 4.0a^2$	1.4	-2	1.4	-2	0.9	-1.3	0.8	-2.1	0.8	-2.1	0.5	-1.4
	$> 4.0a^2$	0.9	-1.3	0.9	-1.3	0.9	-1.3	0.5	-1.4	0.5	-1.4	0.5	-1.4
45°	$\leq a^2$	1.6	-2.2	1.2	-1.7	0.8	-1.1	1	-2.4	0.8	-1.8	0.5	-1.2
	$> a^2, \leq 4.0a^2$	1.2	-1.7	1.2	-1.7	0.8	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$> 4.0a^2$	0.8	-1.1	0.8	-1.1	0.8	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2

Notes:

1. C_N denotes net pressures (contributions from top and bottom surfaces).
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ other than those shown, linear interpolation is permitted.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
6. Notation:
 - a = 10% of least horizontal dimension or $0.4h$, whichever is smaller but not less than 4% of least horizontal dimension or 0.9 m. Dimension "a" is as shown in Figure 207E.8-1.
 - h = mean roof height, m
 - L = Horizontal dimension of building, measured in along wind direction, m
 - θ = angle of plane of roof from horizontal, °

Figure 207E.8-3
Net Pressure Coefficient, C_N Troughed Free Roofs, $\theta \leq 45^\circ$, $0.25 \leq h/L \leq 1.0$
Open Buildings



Windward Parapet

Load Case A

1. Windward parapet pressure (p_1) is determined using the positive wall pressure (p_5) zones 4 or 5 from the applicable figure.
2. Leeward parapet pressure (p_2) is determined using the negative roof pressure (p_7) zones 2 or 3 from the applicable figure.

Leeward Parapet

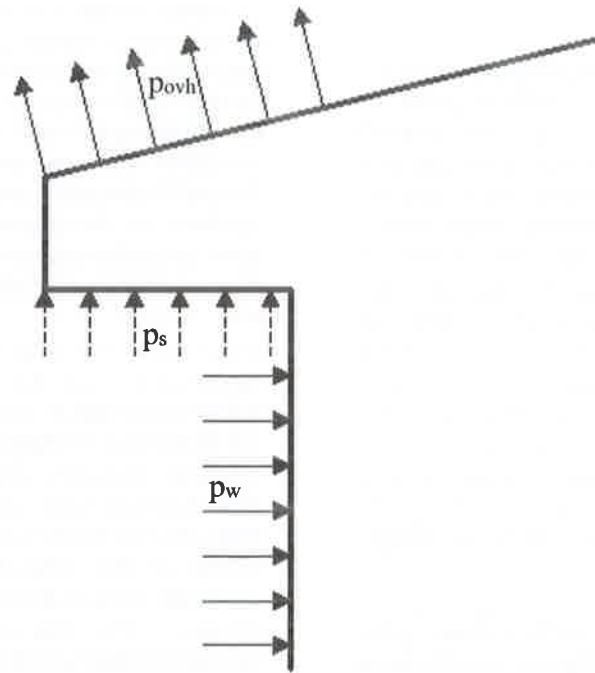
Load Case B

1. Windward parapet pressure (p_3) is determined using the positive wall pressure (p_5) zones 4 or 5 from the applicable figure.
2. Leeward parapet pressure (p_4) is determined using the negative wall pressure (p_6) zones 4 or 5 from the applicable figure.

User Note:

See Note 5 in Figure 207E.4-2A and Note 7 in Figure 207E.6-1 for reductions in component and cladding roof pressures when parapets 0.9 m or higher are present.

Figure 207E.9-1
Parapet Wind Loads C&C – Part 6, All Building Heights All Building Types



Notes:

1. Net roof pressure p_{ovh} on roof overhangs is determined from interior, edge or corner zones as applicable from figures.
2. Net pressure p_{ovh} from figures includes pressure contribution from top and bottom surfaces of roof overhang.
3. Positive pressure at roof overhang soffit p_s is the same as adjacent wall pressure p_w .

Figure 207E.10-1
Wind Loading – Roof Overhangs, C&C, All Building Heights All Building Types

207F Wind Tunnel Procedure

Wind tunnel testing is specified when a structure contains any of the characteristics defined in Sections 207B.1.3, 207C.1.3, 207D.1.3, or 207E.1.3 or when the designer wishes to more accurately determine the wind loads. For some building shapes wind tunnel testing can reduce the conservatism due to enveloping of wind loads inherent in the Directional Procedure, Envelope Procedure, or Analytical Procedure for Components and Cladding. Also, wind tunnel testing accounts for shielding or channeling and can more accurately determine wind loads for a complex building shape than the Directional Procedure, Envelope Procedure, or Analytical Procedure for Components and Cladding. It is the intent of the code that any building or other structure be allowed to use the wind tunnel testing method to determine wind loads. Requirements for proper testing are given in Section 207F.2.

It is common practice to resort to wind tunnel tests when design data are required for the following wind-induced loads:

1. Curtain wall pressures resulting from irregular geometry.
2. Across-wind and/or torsional loads.
3. Periodic loads caused by vortex shedding.
4. Loads resulting from instabilities, such as flutter or galloping.

Boundary-layer wind tunnels capable of developing flows that meet the conditions stipulated in Section 207F.2 typically have test-section dimensions in the following ranges: width of 2 to 4 m, height of 2 to 3 m, and length of 15 to 30 m. Maximum wind speeds are ordinarily in the range of 10 to 45 m/s. The wind tunnel may be either an open-circuit or closed circuit type.

Three basic types of wind-tunnel test models are commonly used. These are designated as follows: (1) rigid Pressure Model (PM), (2) rigid high-frequency base balance model (H-FBBM), and (3) Aero-elastic Model (AM). One or more of the models may be employed to obtain design loads for a particular building or structure. The PM provides local peak pressures for design of elements, such as cladding and mean pressures, for the determination of overall mean loads. The H-FBBM measures overall fluctuating loads (aerodynamic admittance) for the determination of dynamic responses. When motion of a building or structure influences the wind loading, the AM is employed for direct measurement of overall loads, deflections, and

accelerations. Each of these models, together with a model of the surroundings (proximity model), can provide information other than wind loads, such as snow loads on complex roofs, wind data to evaluate environmental impact on pedestrians, and concentrations of air-pollutant emissions for environmental impact determinations. Several references provide detailed information and guidance for the determination of wind loads and other types of design data by wind tunnel tests (Cermak 1977, Reinhold 1982, ASCE 1999, and Boggs and Peterka 1989).

Wind tunnel tests frequently measure wind loads that are significantly lower than required by Sections 207A, 207B, 207C, 207D, and 207E due to the shape of the building, the likelihood that the highest wind speeds occur at directions where the building's shape or pressure coefficients are less than their maximum values, specific buildings included in a detailed proximity model that may provide shielding in excess of that implied by exposure categories, and necessary conservatism in enveloping load coefficients in Sections 207C and 207E. In some cases, adjacent structures may shield the structure sufficiently that removal of one or two structures could significantly increase wind loads. Additional wind tunnel testing without specific nearby buildings (or with additional buildings if they might cause increased loads through channeling or buffeting) is an effective method for determining the influence of adjacent buildings.

For this reason, the code limits the reduction that can be accepted from wind tunnel tests to 80 percent of the result obtained from Part 1 of Section 207B or Part 1 of Section 207C, or Section 207E, if the wind tunnel proximity model included any specific influential buildings or other objects that, in the judgment of an experienced wind engineer, are likely to have substantially influenced the results beyond those characteristic of the general surroundings. If there are any such buildings or objects, supplemental testing can be performed to quantify their effect on the original results and possibly justify a limit lower than 80 percent, by removing them from the detailed proximity model and replacing them with characteristic ground roughness consistent with the adjacent roughness. A specific influential building or object is one within the detailed proximity model that protrudes well above its surroundings, or is unusually close to the subject building, or may otherwise cause substantial sheltering effect or magnification of the wind loads. When these supplemental test results are included with the original results, the acceptable results are then considered to be the higher of both conditions.

However, the absolute minimum reduction permitted is 65 percent of the baseline result for components and cladding, and 50 percent for the main wind force resisting system. A higher reduction is permitted for MWFRS, because

components and cladding loads are more subject to changes due to local channeling effects when surroundings change and can easily be dramatically increased when a new adjacent building is constructed. It is also recognized that cladding failures are much more common than failures of the MWFRS. In addition, for the case of MWFRS it is easily demonstrated that the overall drag coefficient for certain common building shapes, such as circular cylinders especially with rounded or domed tops, is one-half or less of the drag coefficient for the rectangular prisms that form the basis of Section 207B, 207C, and 207E.

For components and cladding, the 80-percent limit is defined by the interior zones 1 and 4 in Figures 207E.4-1, 207E.4-2A, 207E.4-2B, 207E.4-2C, 207E.4-3, 207E.4-4, 207E.4-5A, 207E.4-5B, 207E.4-6, 207E.4-7, and 207E.5-1. This limitation recognizes that pressures in the edge zones are the ones most likely to be reduced by the specific geometry of real buildings compared to the rectangular prismatic buildings assumed in Section 207E. Therefore, pressures in edge and corner zones are permitted to be as low as 80 percent of the interior pressures from Section 207E without the supplemental tests. The 80 percent limit based on zone 1 is directly applicable to all roof areas, and the 80 percent limit based on zone 4 is directly applicable to all wall areas.

The limitation on MWFRS loads is more complex because the load effects (e.g., member stresses or forces, deflections) at any point are the combined effect of a vector of applied loads instead of a simple scalar value. In general the ratio of forces or moments or torques (force eccentricity) at various floors throughout the building using a wind tunnel study will not be the same as those ratios determined from Sections 207B and 207C, and therefore comparison between the two methods is not well defined. Requiring each load effect from a wind tunnel test to be no less than 80 percent of the same effect resulting from Sections 207B and 207C is impractical and unnecessarily complex and detailed, given the approximate nature of the 80 percent value. Instead, the intent of the limitation is effectively implemented by applying it only to a simple index that characterizes the overall loading. For flexible (tall) buildings, the most descriptive index of overall loading is the base overturning moment. For other buildings, the overturning moment can be a poor characterization of the overall loading, and the base shear is recommended instead.

207F.1 Scope

The Wind Tunnel Procedure shall be used where required by Sections 207B.1.3, 207C.1.3, and 207D.1.3. The Wind Tunnel Procedure shall be permitted for any building or structure in lieu of the design procedures specified in Section 207B (MWFRS for buildings of all heights and simple diaphragm buildings with $h \leq 49$ m, Section 207C (MWFRS of low-rise buildings and simple diaphragm low-rise buildings), Section 207D (MWFRS for all other structures), and Section 207E (components and cladding for all building types and other structures).

User Note:

Section 207F may always be used for determining wind pressures for the MWFRS and/or for C&C of any building or structure. This method is considered to produce the most accurate wind pressures of any method specified in this Code.

207F.2 Test Conditions

Wind tunnel tests, or similar tests employing fluids other than air, used for the determination of design wind loads for any building or other structure, shall be conducted in accordance with this section. Tests for the determination of mean and fluctuating forces and pressures shall meet all of the following conditions:

1. The natural atmospheric boundary layer has been modeled to account for the variation of wind speed with height.
2. The relevant macro-(integral) length and micro-length scales of the longitudinal component of atmospheric turbulence are modeled to approximately the same scale as that used to model the building or structure.
3. The modeled building or other structure and surrounding structures and topography are geometrically similar to their full-scale counterparts, except that, for low-rise buildings meeting the requirements of Section 207C.1.2, tests shall be permitted for the modeled building in a single exposure site as defined in Section 207A.7.3.
4. The projected area of the modeled building or other structure and surroundings is less than 8 percent of the test section cross-sectional area unless correction is made for blockage.
5. The longitudinal pressure gradient in the wind tunnel test section is accounted for.

6. Reynolds number effects on pressures and forces are minimized.
7. Response characteristics of the wind tunnel instrumentation are consistent with the required measurements.

207F.3 Dynamic Response

Tests for the purpose of determining the dynamic response of a building or other structure shall be in accordance with Section 207F.2. The structural model and associated analysis shall account for mass distribution, stiffness, and damping.

207F.4 Load Effects

207F.4.1 Mean Recurrence Intervals of Load Effects

The load effect required for Strength Design shall be determined for the same mean recurrence interval as for the Analytical Method, by using a rational analysis method, defined in the recognized literature, for combining the directional wind tunnel data with the directional meteorological data or probabilistic models based thereon. The load effect required for Allowable Stress Design shall be equal to the load effect required for Strength Design divided by 1.6. For buildings that are sensitive to possible variations in the values of the dynamic parameters, sensitivity studies shall be required to provide a rational basis for design recommendations.

Commentary:

Examples of analysis methods for combining directional wind tunnel data with the directional meteorological data or probabilistic models based thereon are described in Lepage and Irwin (1985), Rigato et al. (2001), Isyumov et al. (2003), Irwin et al. (2005), Simiu and Filliben (2005), and Simiu and Miyata (2006).

207F.4.2 Limitations on Wind Speeds

The wind speeds and probabilistic estimates based thereon shall be subject to the limitations described in Section 207A.5.3.

Commentary:

Section 207F.4.2 specifies that the statistical methods used to analyze historical wind speed and direction data for wind tunnel studies shall be subject to the same limitations specified in Section 207F.4.2 that apply to the Analytical Method.

Database-Assisted Design. *Wind-tunnel aerodynamics databases that contain records of pressures measured synchronously at large numbers of locations on the exterior surface of building models have been developed by wind researchers, e.g., Simiu et al. (2003) and Main and Fritz (2006). Such databases include data that permit a designer to determine, without specific wind tunnel tests, wind-induced forces and moments in Main Wind Force Resisting Systems and Components and Cladding of selected shapes and sizes of buildings. A public domain set of such databases, recorded in tests conducted at the University of Western Ontario (Ho et al. 2005 and St. Pierre et al. 2005) for buildings with gable roofs is available on the National Institute of Standards and Technology (NIST) website: www.nist.gov/wind. Interpolation software for buildings with similar shape and with dimensions close to and intermediate between those included in the set of databases is also available on that site. Because the database results are for generic surroundings as permitted in item 3 of Section 207F.2, interpolation or extrapolation from these databases should be used only if condition 2 of Section 207B.1.2 is true. Extrapolations from available building shapes and sizes are not permitted, and interpolations in some instances may not be advisable. For these reasons, the guidance of an engineer experienced in wind loads on buildings and familiar with the usage of these databases is recommended. All databases must have been obtained using testing methodology that meets the requirements for wind tunnel testing specified in Section 207F.*

207F.4.3 Limitations on Loads

Loads for the main wind force resisting system determined by wind tunnel testing shall be limited such that the overall principal loads in the x and y directions are not less than 80 percent of those that would be obtained from Part 1 of Section 207B or Part 1 of Section 207C. The overall principal load shall be based on the overturning moment for flexible buildings and the base shear for other buildings.

Pressures for components and cladding determined by wind tunnel testing shall be limited to not less than 80 percent of those calculated for Zone 4 for walls and Zone 1 for roofs using the procedure of Section 207E. These Zones refer to those shown in Figures 207E.4-1, 207E.4-2A, 207E.4-2B, 207E.4-2C, 207E.4-3, 207E.4-4, 207E.4-5A, 207E.4-5B, 207E.4-6, 207E.4-7, and 207E.6-1.

The limiting values of 80 percent may be reduced to 50 percent for the main wind force resisting system and 65 percent for components and cladding if either of the following conditions applies:

1. There were no specific influential buildings or objects within the detailed proximity model.
2. Loads and pressures from supplemental tests for all significant wind directions in which specific influential buildings or objects are replaced by the roughness representative of the adjacent roughness condition, but not rougher than exposure B, are included in the test results.

207F.5 Wind-Borne Debris

Glazing in buildings in wind-borne debris regions shall be protected in accordance with Section 207A.10.3.

SECTION 208 EARTHQUAKE LOADS

208.1 General

208.1.1 Purpose

The purpose of the succeeding earthquake provisions is primarily to design seismic-resistant structures to safeguard against major structural damage that may lead to loss of life and property. These provisions are not intended to assure zero-damage to structures nor maintain their functionality after a severe earthquake.

208.1.2 Minimum Seismic Design

Structures and portions thereof shall, as a minimum, be designed and constructed to resist the effects of seismic ground motions as provided in this section.

208.1.3 Seismic and Wind Design

When the code-prescribed wind design produces greater effects, the wind design shall govern, but detailing requirements and limitations prescribed in this section and referenced sections shall be made to govern.

208.2 Definitions

See Section 202.

208.3 Symbols and Notations

- A_B = ground floor area of structure to include area covered by all overhangs and projections, m^2
 A_c = the combined effective area of the shear walls in the first storey of the structure, m^2
 A_e = the minimum cross-sectional area in any horizontal plane in the first storey of a shear wall, m^2
 A_x = the torsional amplification factor at Level x
 a_p = numerical coefficient specified in Section 208.7 and set forth in Table 208-13
 C_a = seismic coefficient, as set forth in Table 208-7
 C_t = numerical coefficient given in Section 208.5.2.2
 C_v = seismic coefficient, as set forth in Table 208-8
 D = dead load
 D_e = the length of a shear wall in the first storey in the direction parallel to the applied forces, m
 E, E_h, E_m, E_v = earthquake loads set forth in Section 208.6
 F_x = design seismic force applied to Level i, n or x , respectively
 F_p = design seismic force on a part of the structure
 F_{px} = design seismic force on a diaphragm

- F_t = that portion of the base shear, V , considered concentrated at the top of the structure in addition to F_n
 f_i = lateral force at Level i for use in Equation 208-14
 g = acceleration due to gravity = 9.815 m/sec^2
 h_i, h_n, h_x = height above the base to Level i, n or x , respectively, m
 I = importance factor given in Table 208-1
 I_p = importance factor for nonstructural component as given in Table 208-1
 L = live load
Level i = level of the structure referred to by the subscript i
“ $i = 1$ ” designates the first level above the base
Level n = that level that is uppermost in the main portion of the structure
Level x = that level that is under design consideration
“ $x = 1$ ” designates the first level above the base
 M = maximum moment magnitude
 N_a = near-source factor used in the determination of C_a in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes as set forth in Tables 208-4 and 208-5
 N_v = near-source factor used in the determination of C_v in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes as set forth in Tables 208-4 and 208-6
 PI = plasticity index of soil determined in accordance with approved national standards
 R = numerical coefficient representative of the inherent over-strength and global ductility capacity of lateral-force-resisting systems, as set forth in Table 208-11 or 208-12
 r = a ratio used in determining ρ , the redundancy/reliability factor. See Section 208.5.
 $S_A, S_B, S_C, S_D, S_E, S_F$ = soil profile types as set forth in Table 208-2
 T = elastic fundamental period of vibration of the structure in the direction under consideration, sec
 V = base shear given by Equations. 208-8, 208-9, 208-10, 208-11 or 208-15
 V_x = the design storey shear in Storey x
 W = the total seismic dead load defined in Section 208.5.2.1
 w_i, w_x = that portion of W located at or assigned to Level i or x , respectively
 W_p = the weight of an element or component
 w_{px} = the weight of the diaphragm and the element tributary thereto at Level x , including applicable portions of other loads defined in Section 208.6.1
 Z = seismic zone factor as given in Table 208-3

- Δ_M = Maximum Inelastic Response Displacement, which is the total drift or total storey drift that occurs when the structure is subjected to the Design Basis Ground Motion, including estimated elastic and inelastic contributions to the total deformation defined in Section 208.6.4.2, mm
- Δ_S = Design Level Response Displacement, which is the total drift or total storey drift that occurs when the structure is subjected to the design seismic forces, mm
- δ_i = horizontal displacement at Level i relative to the base due to applied lateral forces, f_i , for use in Equation 208-14, mm
- ρ = Redundancy/Reliability Factor given by Equation 208-20
- Ω_o = Seismic Force Amplification Factor, which is required to account for structural over-strength and set forth in Table 208-11

208.4 Basis for Design

208.4.1 General

The procedures and the limitations for the design of structures shall be determined considering seismic zoning, site characteristics, occupancy, configuration, structural system and height in accordance with this section. Structures shall be designed with adequate strength to withstand the lateral displacements induced by the Design Basis Ground Motion, considering the inelastic response of the structure and the inherent redundancy, over-strength and ductility of the lateral force-resisting system.

The minimum design strength shall be based on the Design Seismic Forces determined in accordance with the static lateral force procedure of Section 208.5, except as modified by Section 208.5.3.5.4.

Where strength design is used, the load combinations of Section 203.3 shall apply. Where Allowable Stress Design is used, the load combinations of Section 203.4 shall apply.

Allowable Stress Design may be used to evaluate sliding or overturning at the soil-structure interface regardless of the design approach used in the design of the structure, provided load combinations of Section 203.4 are utilized.

208.4.2 Occupancy Categories

For purposes of earthquake-resistant design, each structure shall be placed in one of the occupancy categories listed in Table 103-1. Table 208-1 assigns importance factors, I and I_p , and structural observation requirements for each category.

Table 208-1 - Seismic Importance Factors

Occupancy Category ¹	Seismic Importance Factor, I	Seismic Importance Factor, I_p ²
I. Essential Facilities ³	1.50	1.50
II. Hazardous Facilities	1.25	1.50
III. Special Occupancy Structures	1.00	1.00
IV. Standard Occupancy Structures	1.00	1.00
V. Miscellaneous structures	1.00	1.00

¹ See Table 103-1 for occupancy category listing

² The limitation of I_p for panel connections in Section 208.7.2.3 shall be **1.0** for the entire connector

³ Structural observation requirements are given in Section 107.9

⁴ For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as **1.5**

208.4.3 Site Geology and Soil Characteristics

Each site shall be assigned a soil profile type based on properly substantiated geotechnical data using the site categorization procedure set forth in Section 208.4.3.1.1 and Table 208-2.

Exception:

When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used. Soil Profile Type S_E or S_F need not be assumed unless the building official determines that Type S_E or S_F may be present at the site or in the event that Type S_E or S_F is established by geotechnical data.

208.4.3.1 Soil Profile Type

Soil Profile Types S_A , S_B , S_C , S_D and S_E are defined in Table 208-2 and Soil Profile Type S_F is defined as soils requiring site-specific evaluation as follows:

1. Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.
2. Peats and/or highly organic clays, where the thickness of peat or highly organic clay exceeds 3.0 m.
3. Very high plasticity clays with a plasticity index, $PI > 75$, where the depth of clay exceeds 7.5 m.
4. Very thick soft/medium stiff clays, where the depth of clay exceeds 35 m.

5. The criteria set forth in the definition for Soil Profile Type S_F requiring site-specific evaluation shall be considered. If the site corresponds to these criteria, the site shall be classified as Soil Profile Type S_F and a site-specific evaluation shall be conducted.

208.4.3.1.1 Site Categorization Procedure

208.4.3.1.1.1 Scope

This section describes the procedure for determining Soil Profile Types S_A through S_F as defined in Table 208-2.

Table 208-2 - Soil Profile Types

Soil Profile Type	Soil Profile Name / Generic Description	Average Soil Properties for Top 30 m of Soil Profile		
		Shear Wave Velocity, V_s (m/s)	SPT, N (blows/300 mm)	Undrained Shear Strength, S_u (kPa)
S_A	Hard Rock	> 1500		
S_B	Rock	760 to 1500		
S_C	Very Dense Soil and Soft Rock	360 to 760	> 50	> 100
S_D	Stiff Soil Profile	180 to 360	15 to 50	50 to 100
S_E^1	Soft Soil Profile	< 180	< 15	< 50
S_F	Soil Requiring Site-specific Evaluation. See Section 208.4.3.1			

¹ Soil Profile Type S_E also includes any soil profile with more than 3.0 m of soft clay defined as a soil with plasticity index, $PI > 20$, $w_{mc} \geq 40\%$ and $s_u < 24$ kPa. The Plasticity Index, PI , and the moisture content, w_{mc} , shall be determined in accordance with approved national standards.

208.4.3.1.1.2 Definitions

Soil profile types are defined as follows:

- S_A Hard rock with measured shear wave velocity, $v_s > 1500$ m/s
- S_B Rock with 760 m/s $< v_s \leq 1500$ m/s
- S_C Very dense soil and soft rock with 360 m/s $< v_s < 760$ m/s or with either $N > 50$ or $s_u \geq 100$ kPa
- S_D Stiff soil with 180 m/s $\leq v_s \leq 360$ m/s or with $15 \leq N \leq 50$ or 50 kPa $\leq s_u \leq 100$ kPa

- S_E A soil profile with $v_s < 180$ m/s or any profile with more than 3 m of soft clay defined as soil with $PI > 20$, $w_{mc} \geq 40$ percent and $s_u < 25$ kPa
- S_F Soils requiring site-specific evaluation, refer to Section 208.4.3.1

208.4.3.1.1.2.1 v_s , Average Shear Wave Velocity

v_s shall be determined in accordance with the following equation:

$$v_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}} \quad (208-1)$$

where

d_i = thickness of Layer i , m
 v_{si} = shear wave velocity in Layer i , m/s

208.4.3.1.1.2.2 N , Average Field Standard Penetration Resistance and N_{ch} , Average Standard Penetration Resistance for Cohesionless Soil Layers

N and N_{ch} shall be determined in accordance with the following equation:

$$N = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (208-2)$$

$$N_{ch} = \frac{d_s}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (208-3)$$

where

d_i = thickness of Layer i in mm
 d_s = the total thickness of cohesionless soil layers in the top 30 m
 N_i = the standard penetration resistance of soil layer in accordance with approved nationally recognized standards

208.4.3.1.1.2.3 s_u , Average Undrained Shear Strength

s_u shall be determined in accordance with the following equation:

$$s_u = \frac{d_c}{\sum_{i=1}^n \frac{d_i}{s_{ui}}} \quad (208-4)$$

where

d_c = the total thickness ($100 - d_s$) of cohesive soil layers in the top 30 m
 s_{ui} = the undrained shear strength in accordance with approved nationally recognized standards, not to exceed 250 kPa

208.4.3.1.1.2.4 Rock Profiles, S_A and S_B

The shear wave velocity for rock, Soil Profile Type S_B , shall be either measured on site or estimated by a geotechnical engineer, engineering geologist or seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Soil Profile Type S_C .

The hard rock, Soil Profile Type S_A , category shall be supported by shear wave velocity measurement either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 30 m, surficial shear wave velocity measurements may be extrapolated to assess v_s . The rock categories, Soil Profile Types S_A and S_B , shall not be used if there is more than 3 meters of soil between the rock surface and the bottom of the spread footing or mat foundation.

The definitions presented herein shall apply to the upper 30 m of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number from 1 to n at the bottom, where there are a total of n distinct layers in the upper 30 m. The symbol i then refer to any one of the layers between 1 and n .

208.4.3.1.1.2.5 Soft Clay Profile, S_E

The existence of a total thickness of soft clay greater than 3 m shall be investigated where a soft clay layer is defined by $s_u < 24 \text{ kPa}$, $w_{mc} \geq 40$ percent and $PI > 20$. If these criteria are met, the site shall be classified as Soil Profile Type S_E .

208.4.3.1.1.2.6 Soil Profiles S_C , S_D and S_E

Sites with Soil Profile Types S_C , S_D and S_E shall be classified by using one of the following three methods with v_s , N and s_u computed in all cases as specified in Section 208.4.3.1.1.2.

1. v_s for the top 30 meters (v_s method).
2. N for the top 30 meters (N method).
3. N_{CH} for cohesionless soil layers ($PI < 20$) in the top 30 m and average s_u for cohesive soil layers ($PI > 20$) in the top 30 m (s_u method).

208.4.4 Site Seismic Hazard Characteristics

Seismic hazard characteristics for the site shall be established based on the seismic zone and proximity of the site to active seismic sources, site soil profile characteristics and the structure's importance factor.

208.4.4.1 Seismic Zone

The Philippine archipelago is divided into two seismic zones only. Zone 2 covers the provinces of Palawan (except Busuanga), Sulu and Tawi-Tawi while the rest of the country is under Zone 4 as shown in Figure 208-1. Each structure shall be assigned a seismic zone factor Z , in accordance with Table 208-3.

Table 208-3 Seismic Zone Factor Z

ZONE	2	4
Z	0.20	0.40

208.4.4.2 Seismic Source Types

Table 208-4 defines the types of seismic sources. The location and type of seismic sources to be used for design shall be established based on approved geological data; see Figure 208-2A. Type A sources shall be determined from Figure 208-2B, 2C, 2D, 2E or the most recent mapping of active faults by the Philippine Institute of Volcanology and Seismology (PHIVOLCS).

Table 208-4 - Seismic Source Types ¹

Seismic Source Type	Seismic Source Description	Seismic Source Definition
		Maximum Moment Magnitude, M
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity.	$7.0 \leq M \leq 8.4$
B	All faults other than Types A and C.	$6.5 \leq M < 7.0$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.	$M < 6.5$

¹Subduction sources shall be evaluated on a site-specific basis.

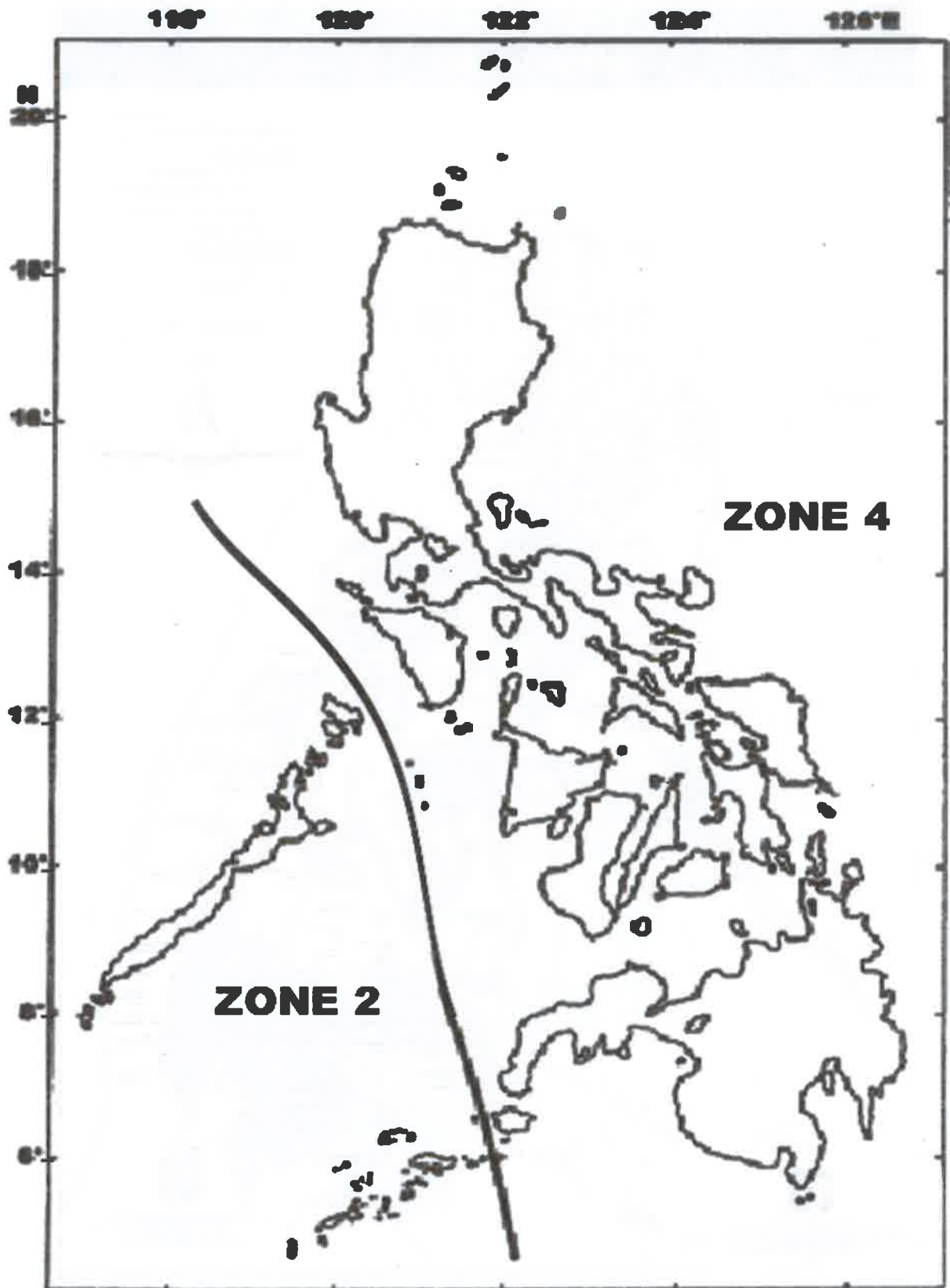


Figure 208-1 Referenced Seismic Map of the Philippines
National Structural Code of the Philippines Volume I, 7th Edition, 2015

Distribution of Active Faults and Trenches in the Philippines

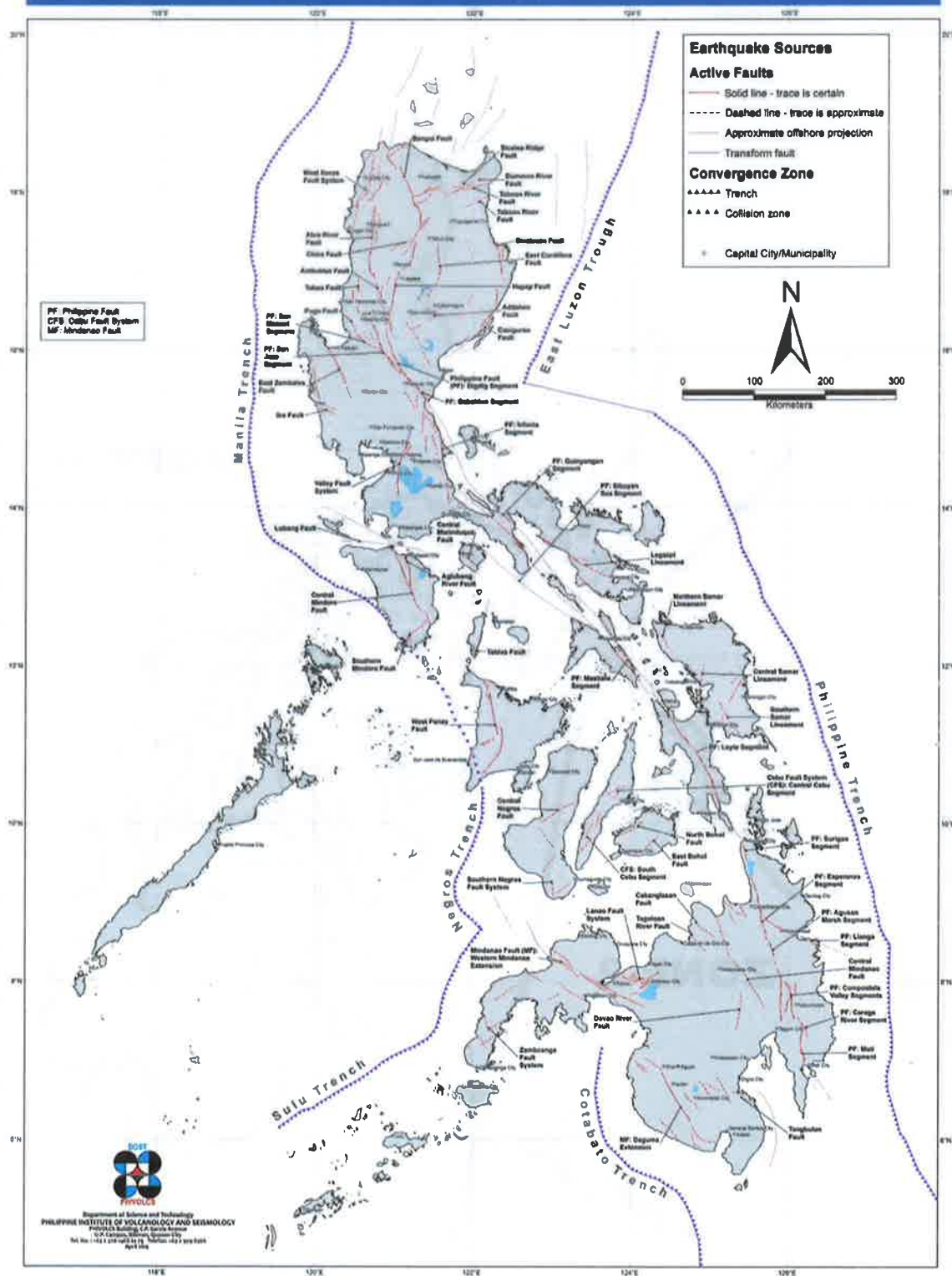


Figure 208-2A Distribution of Active Faults and Trenches in the Philippines

Association of Structural Engineers of the Philippines, Inc. (ASEP)

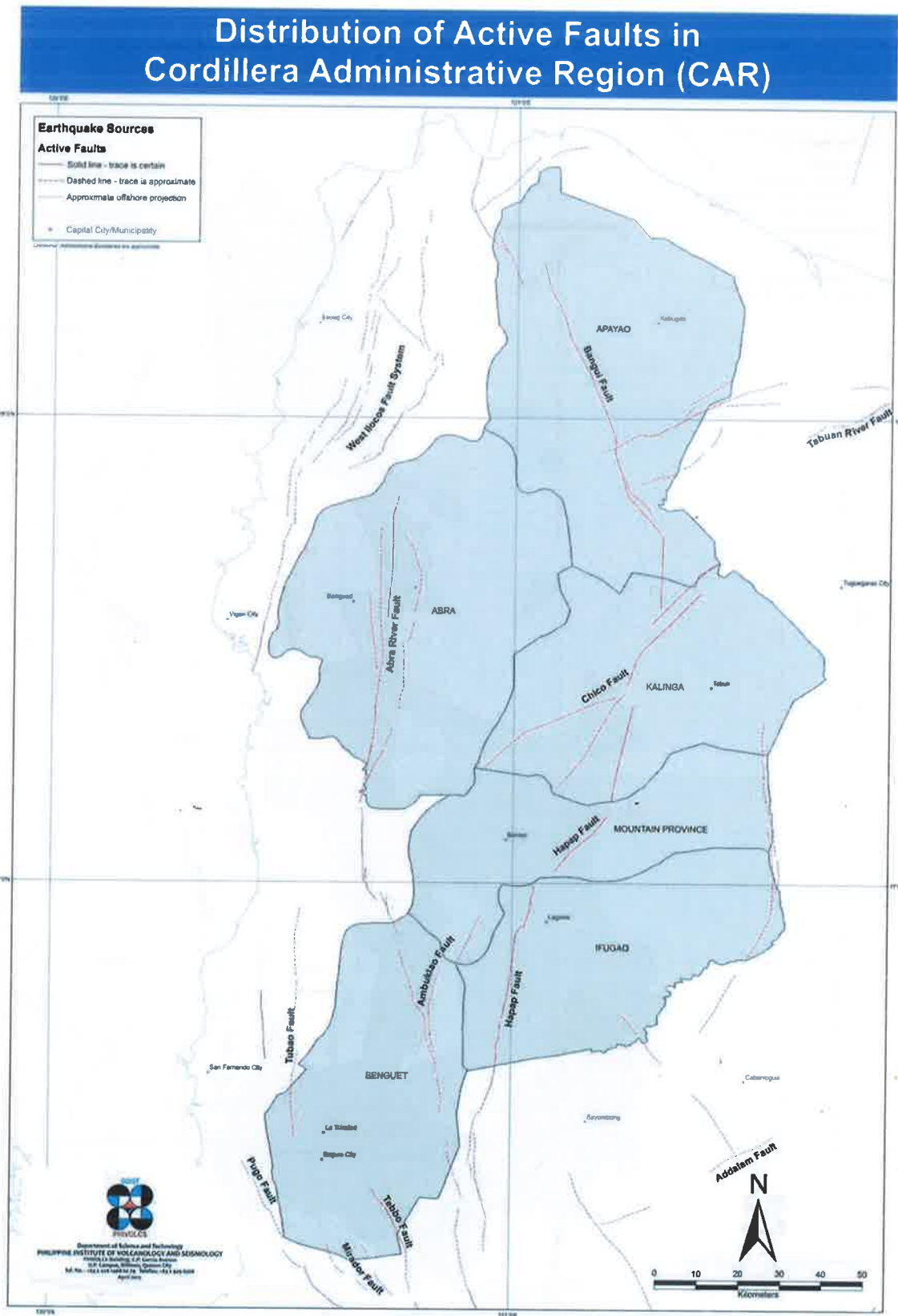


Figure 208-2B Distribution of Active Faults in Cordillera Administrative Region (CAR)
National Structural Code of the Philippines Volume I, 7th Edition, 2015

CHAPTER 2 – Minimum Design Loads



Figure 208-2C Distribution of Active Faults and Trenches in Region 1

Association of Structural Engineers of the Philippines, Inc. (ASEP)

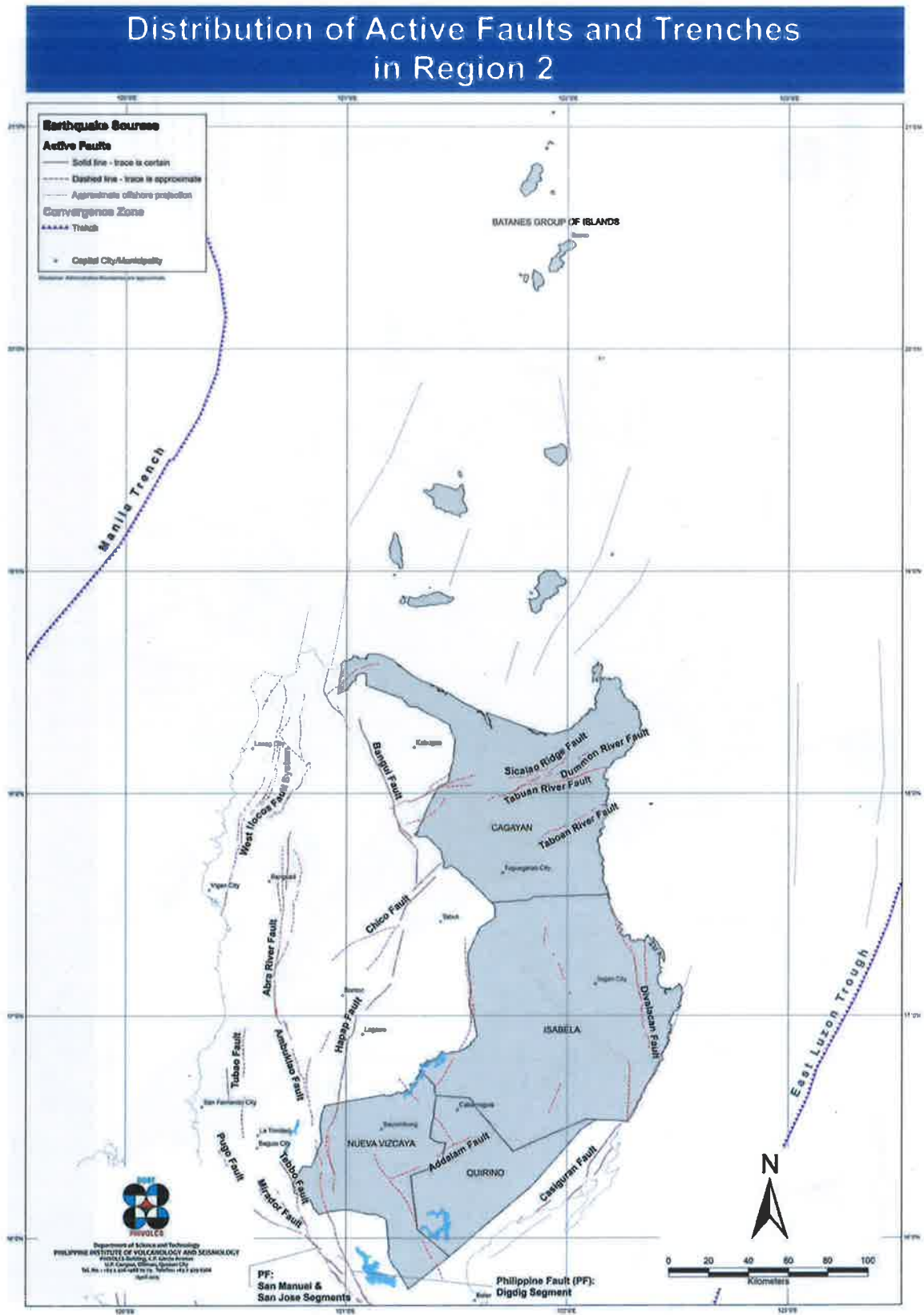


Figure 208-2D Distribution of Active Faults and Trenches in Region 2
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Figure 208-2E Distribution of Active Faults in Region 3

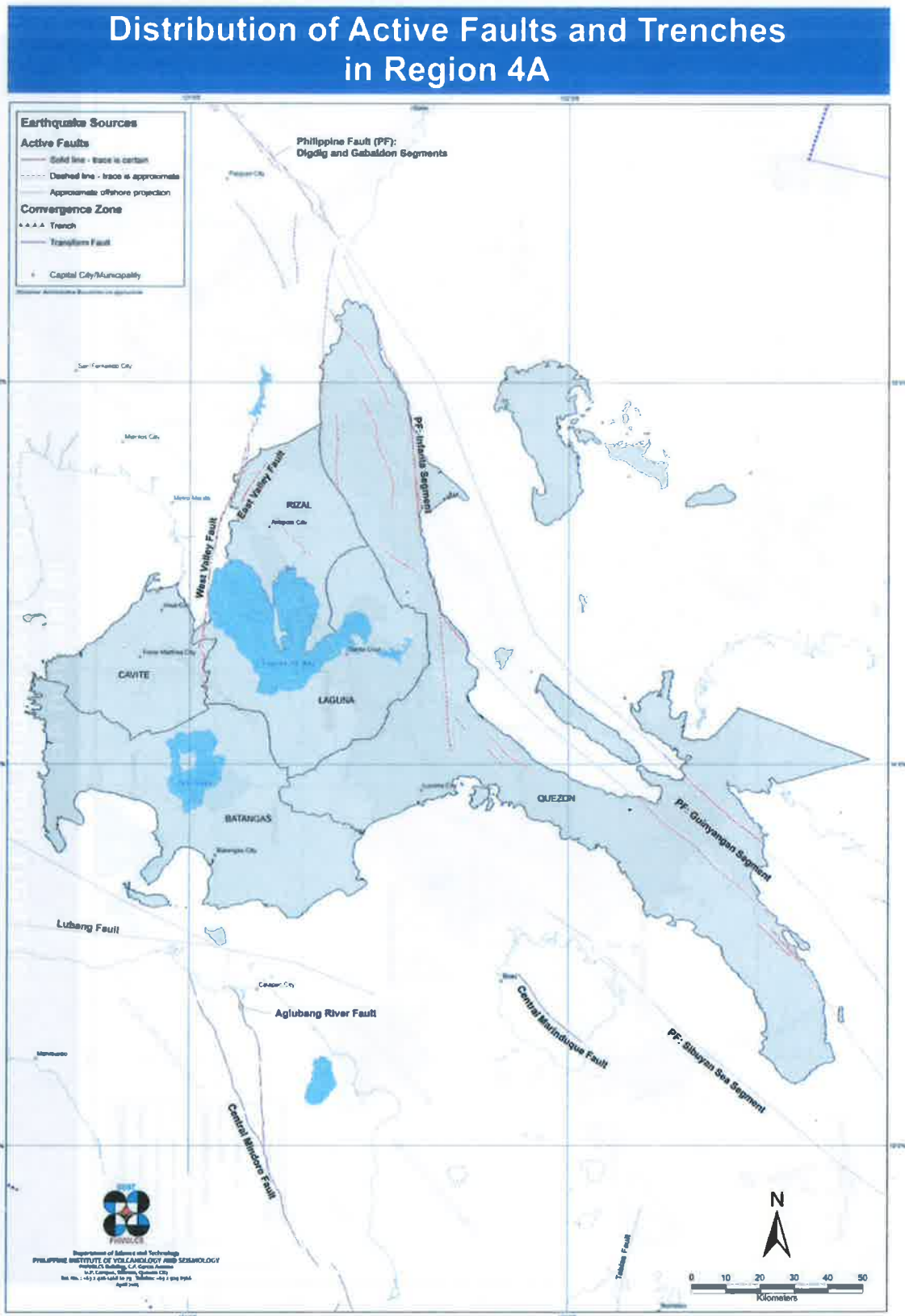
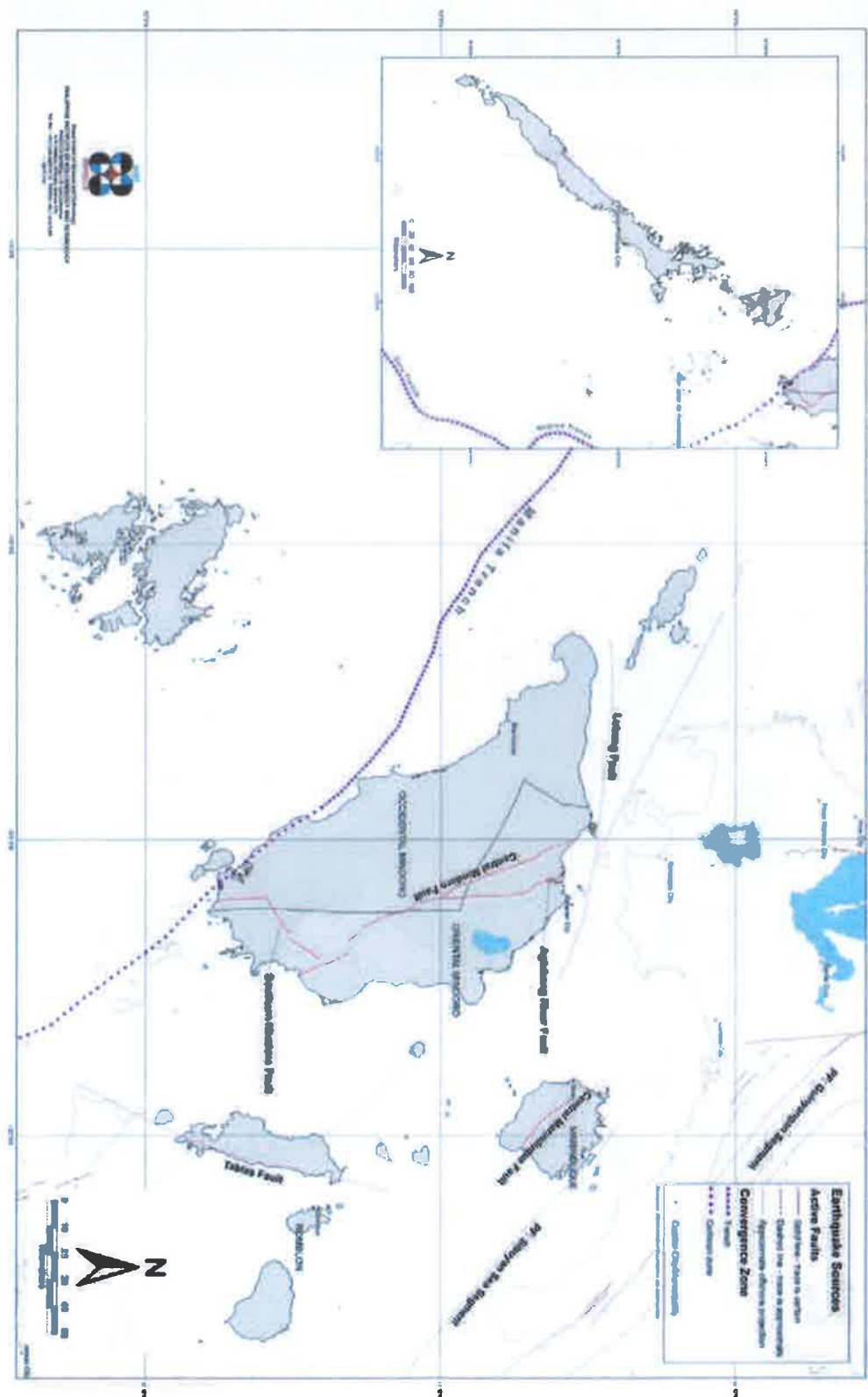


Figure 208-2F Distribution of Active Faults and Trenches in Region 4A

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Distribution of Active Faults and Trenches
in Region 4B

Figure 208-2F Distribution of Active Faults and Trenches in Region 4B

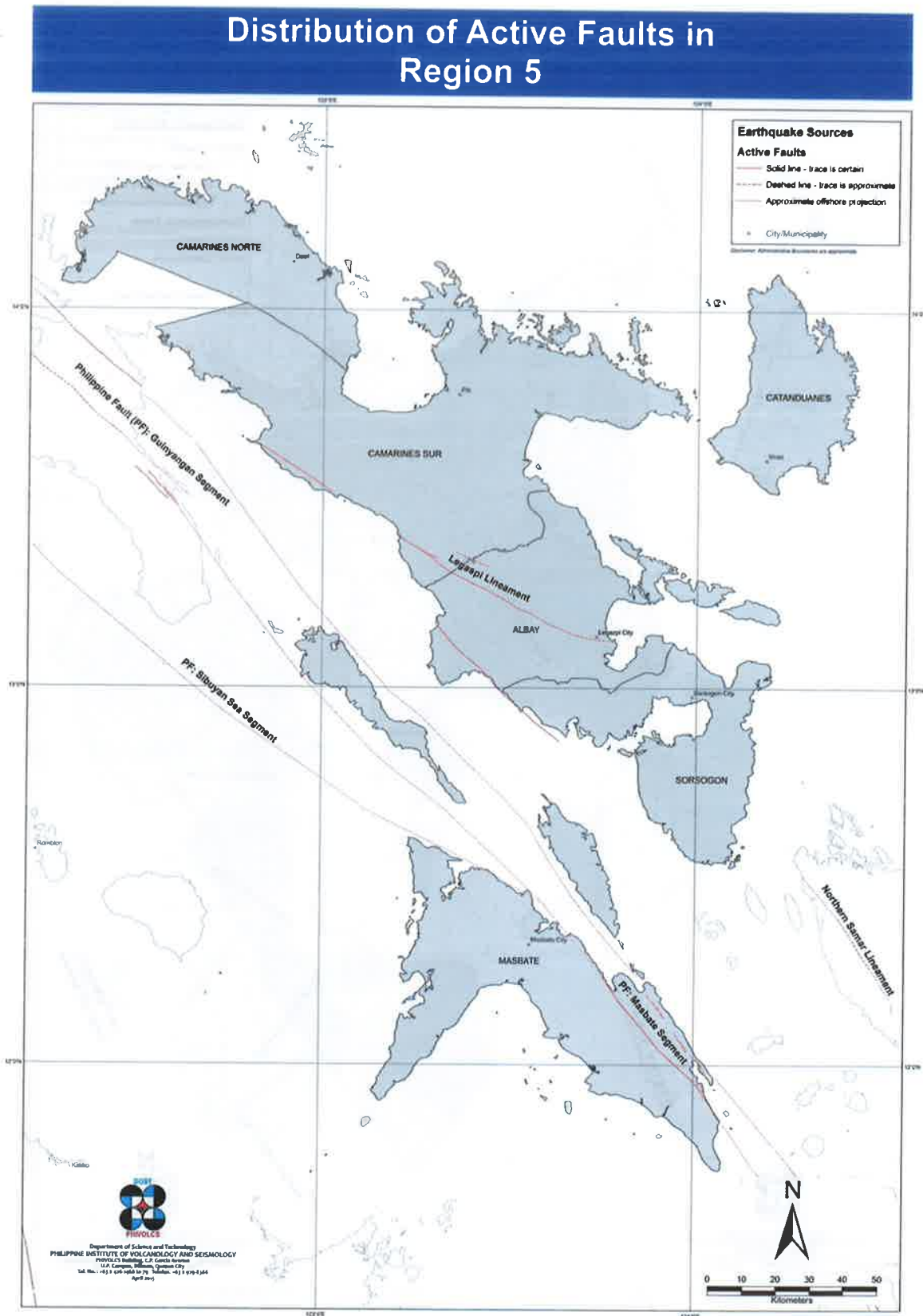


Figure 208-2E Distribution of Active Faults in Region 5

Distribution of Active Faults and Trenches in Region 6

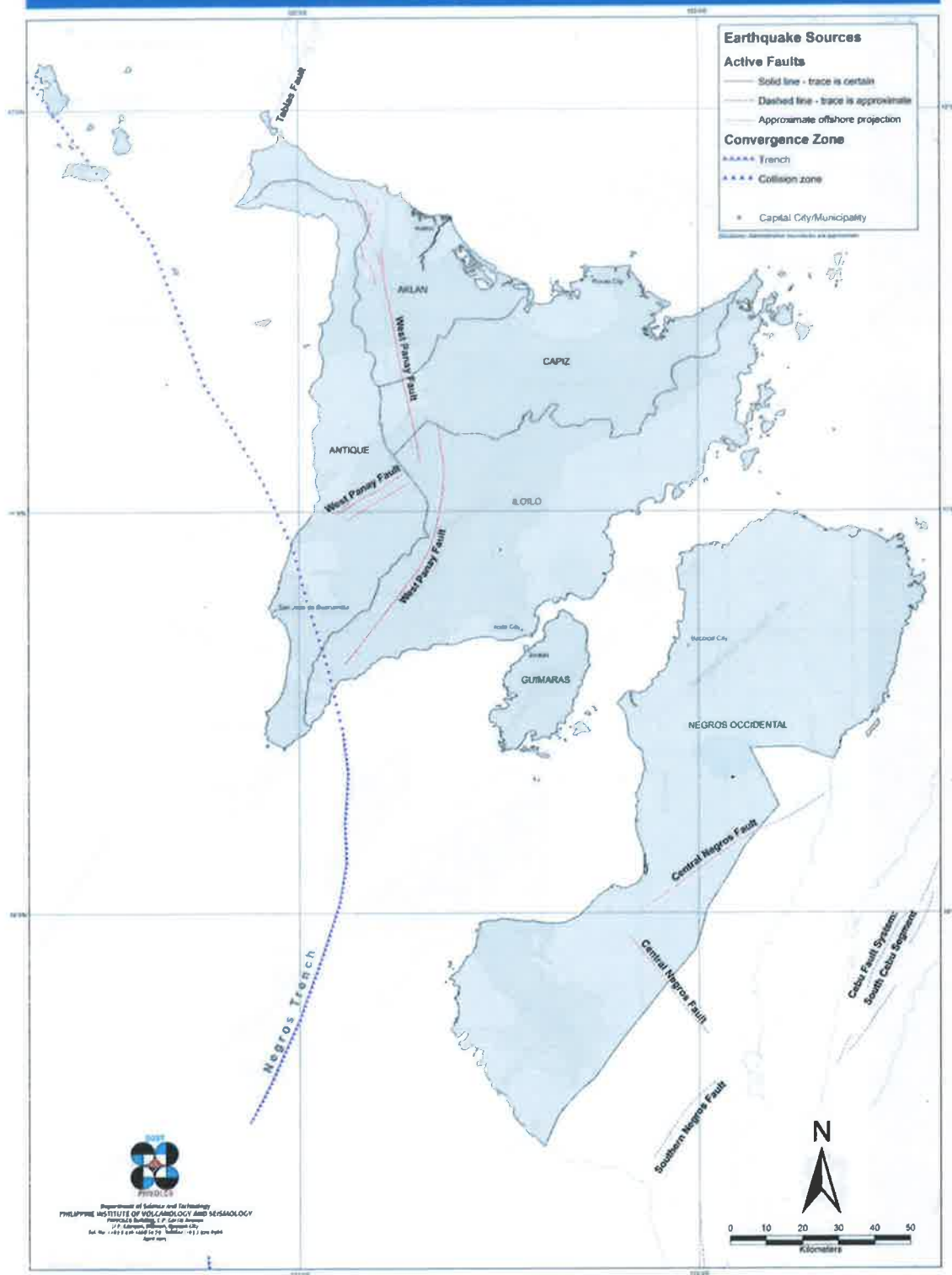


Figure 208-2F Distribution of Active Faults and Trenches in Region 6

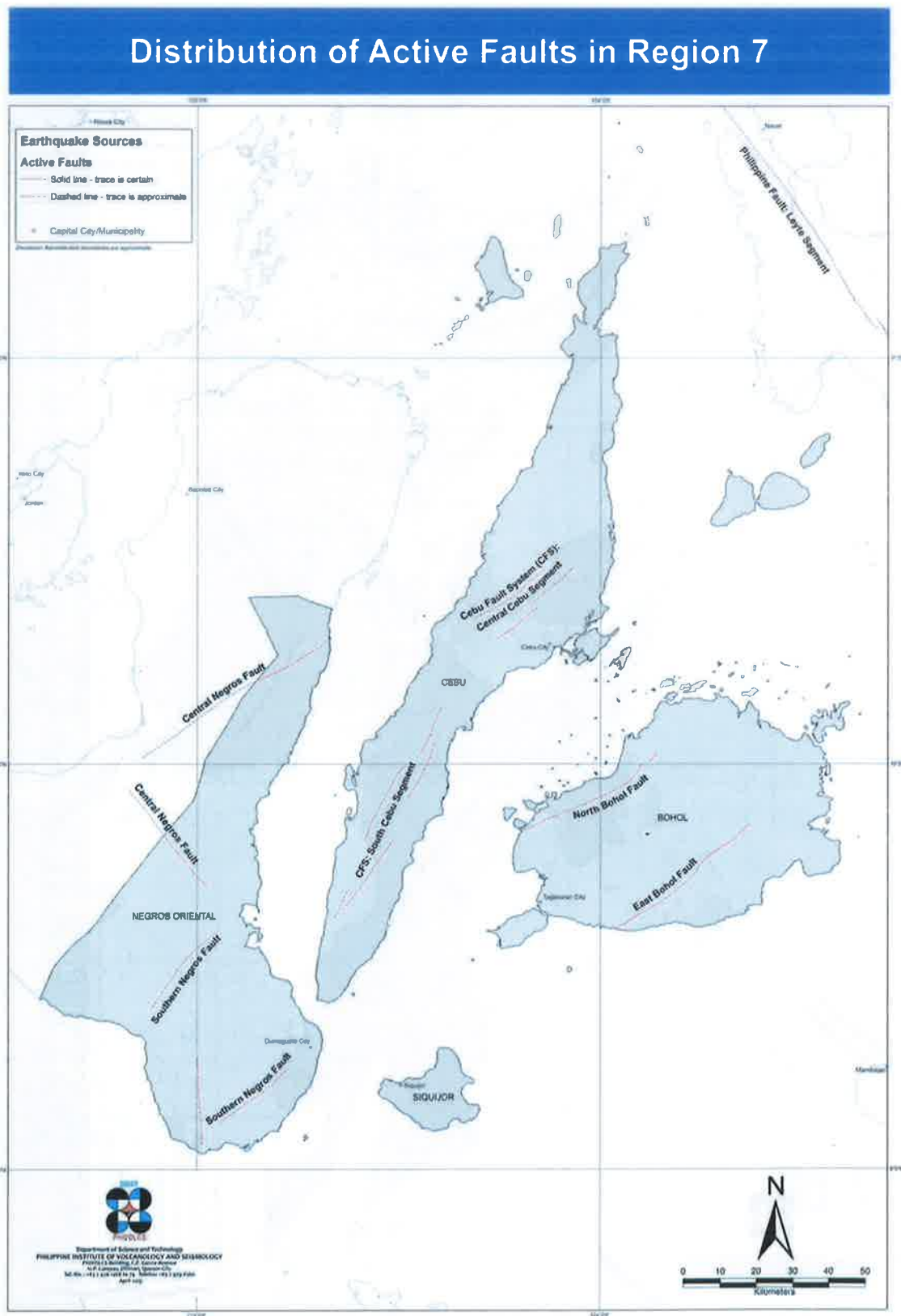


Figure 208-2G Distribution of Active Faults in Region 7

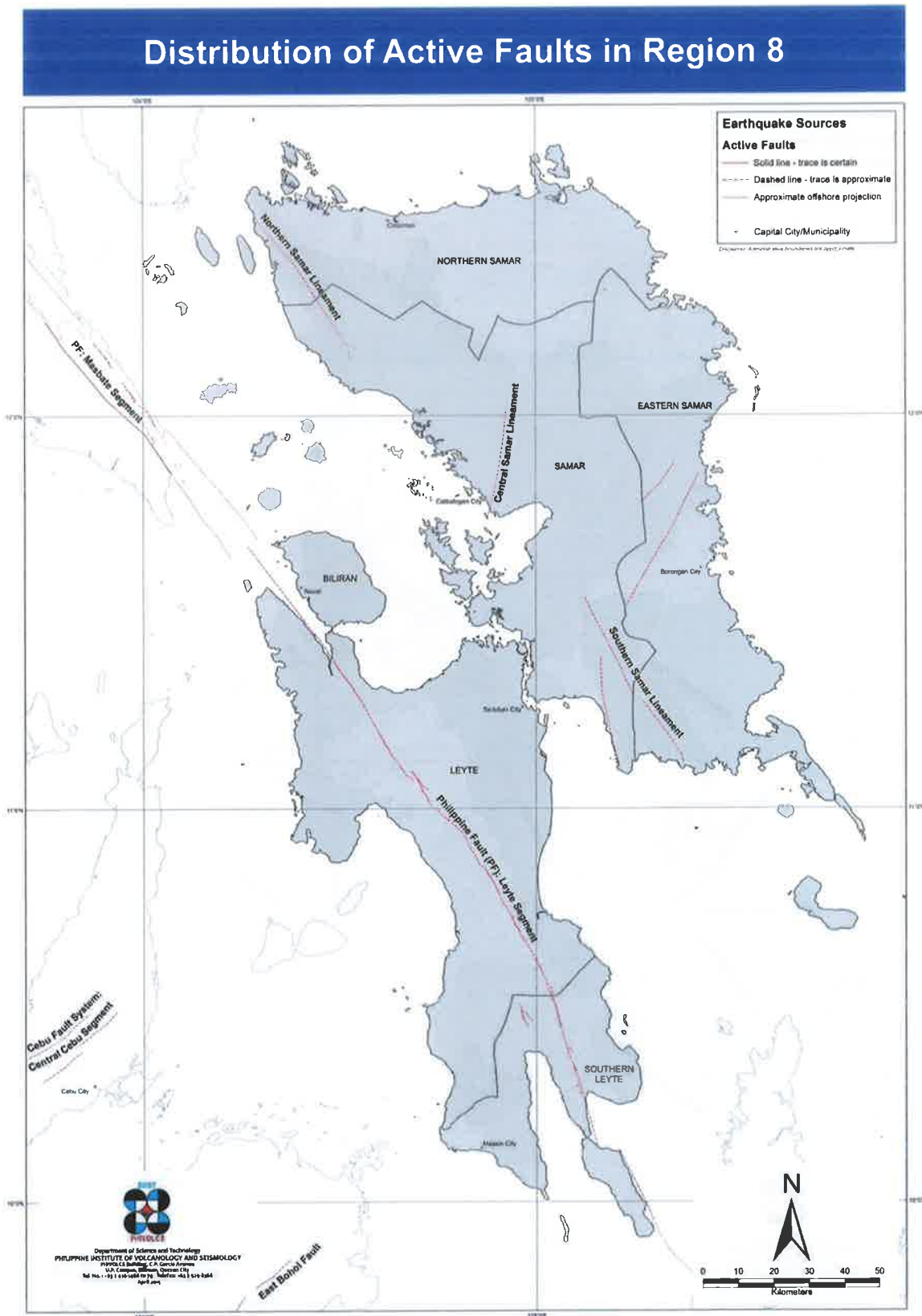


Figure 208-2H Distribution of Active Faults in Region 8

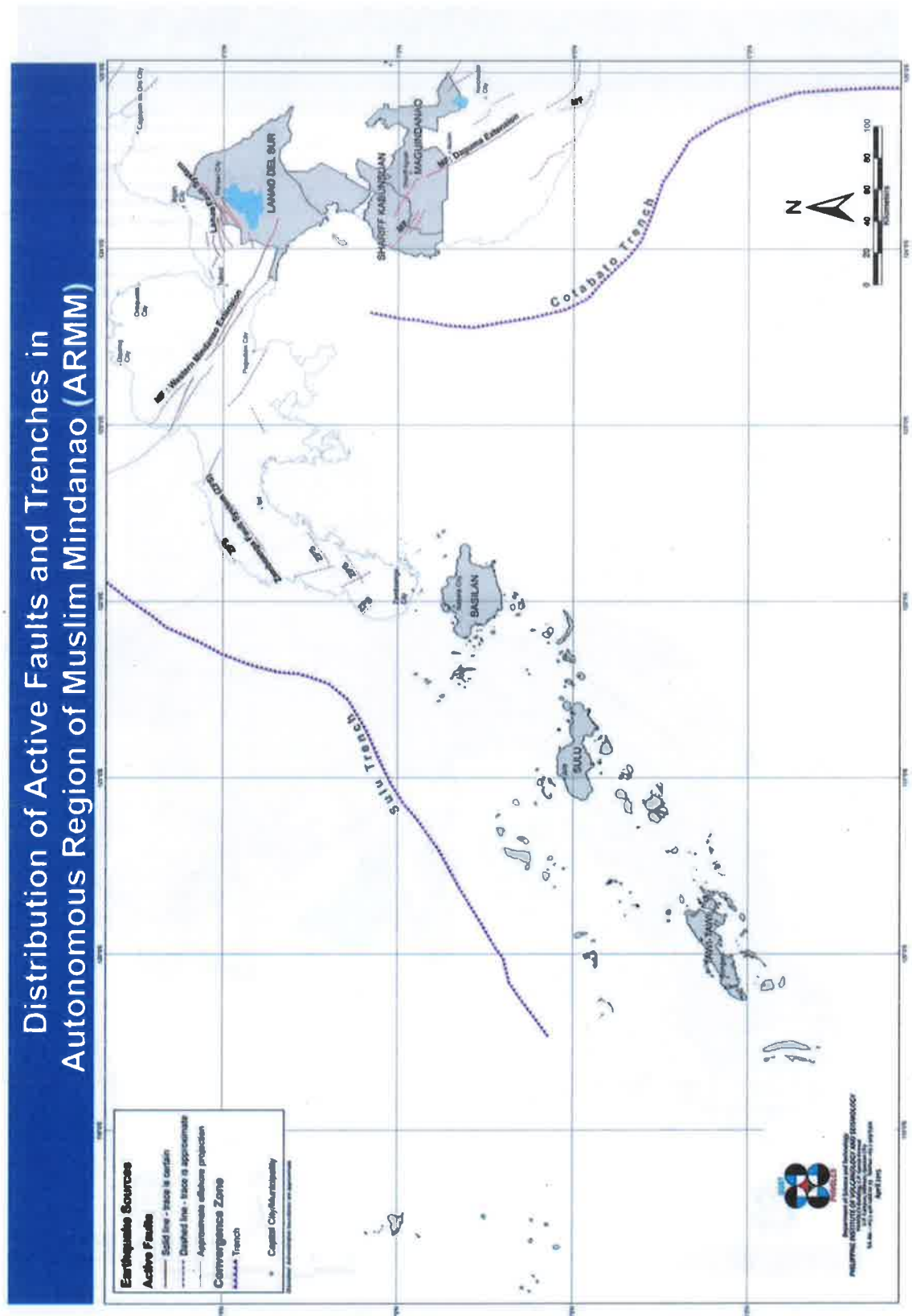


Figure 208-2I Distribution of Active Faults and Trenches in Autonomous Region of Muslim Mindanao (ARMM)

Distribution of Active Faults and Trenches in Region 9

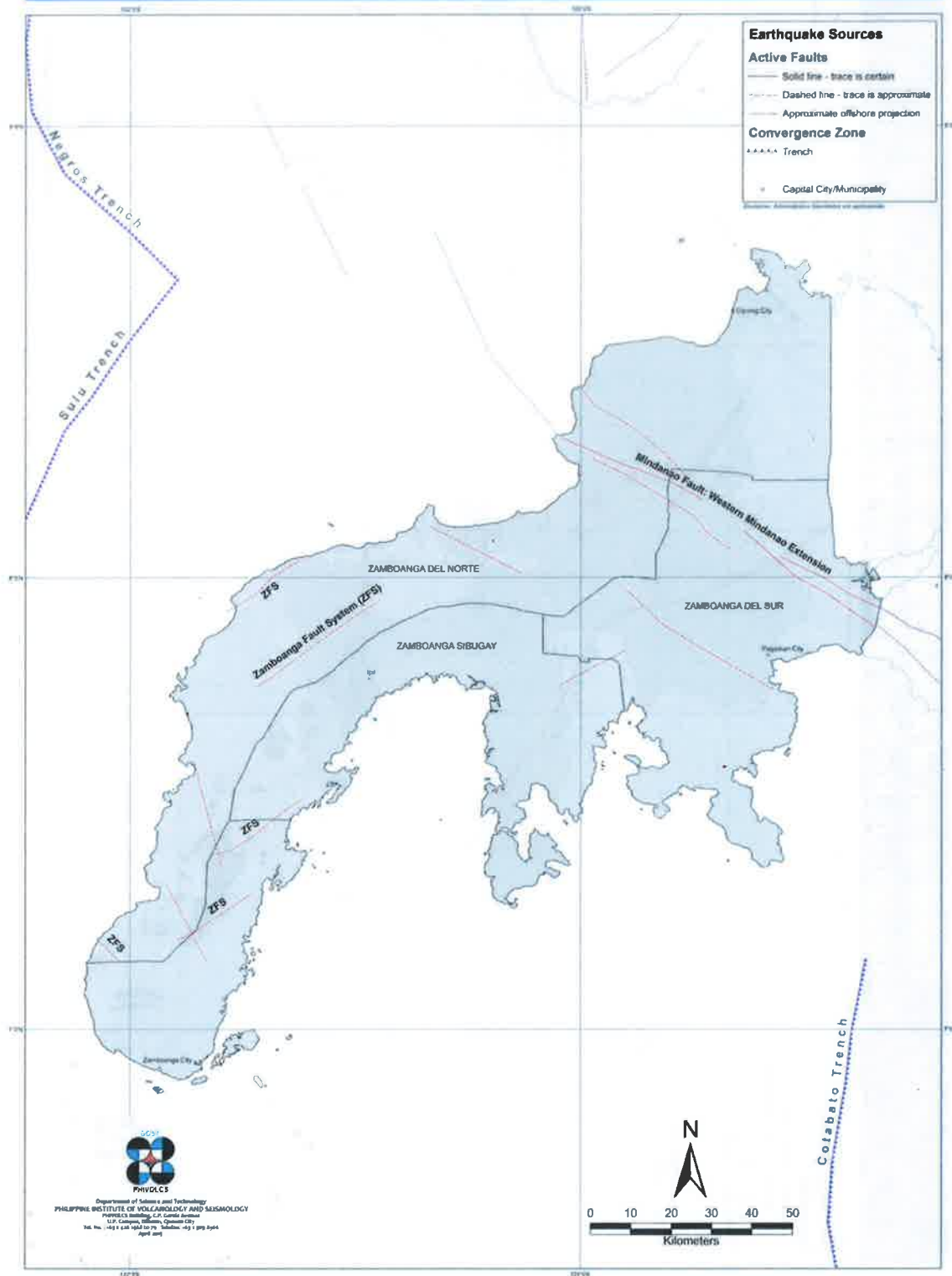


Figure 208-2J Distribution of Active Faults and Trenches in Region 9

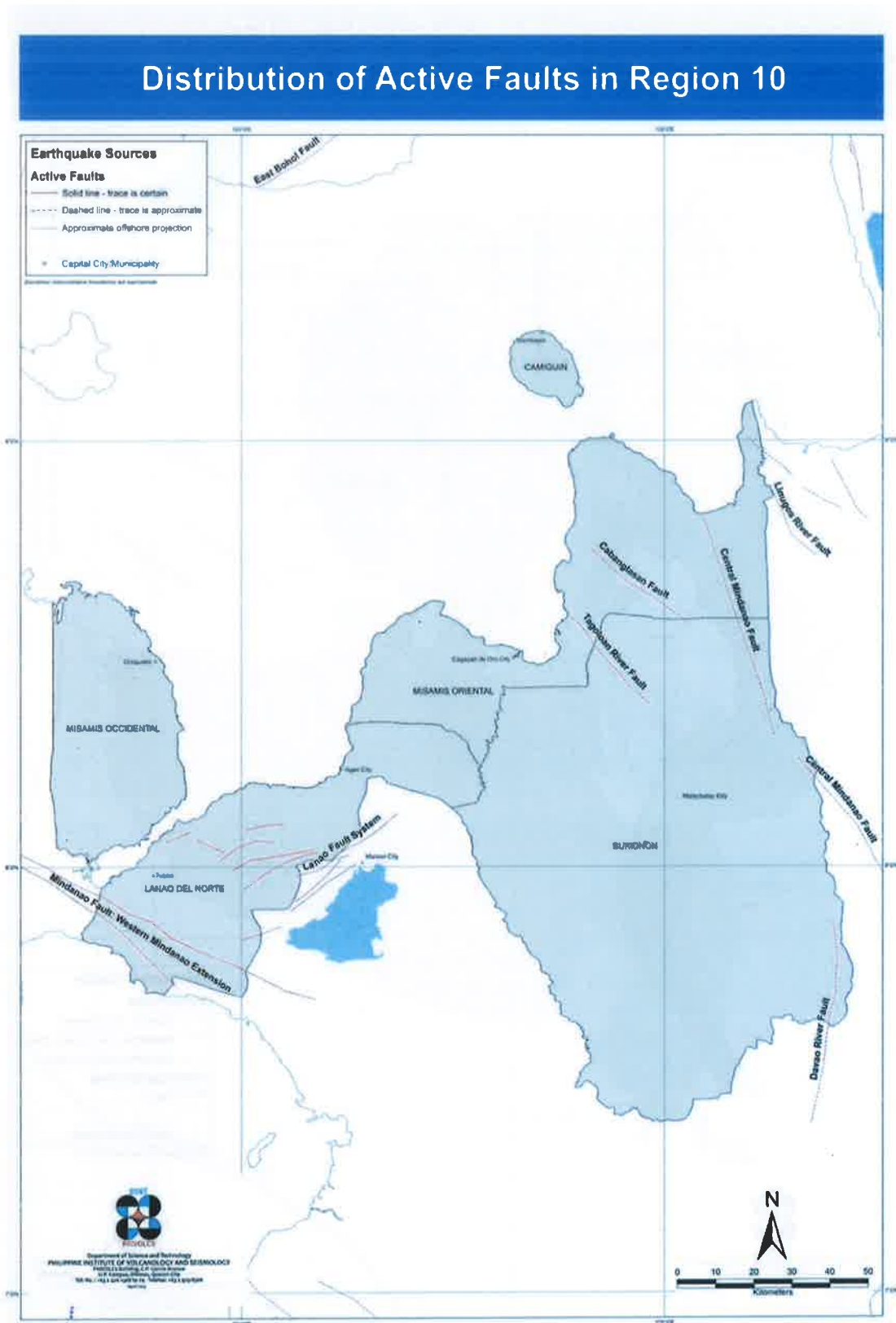


Figure 208-2K Distribution of Active Faults and Trenches in Region 10

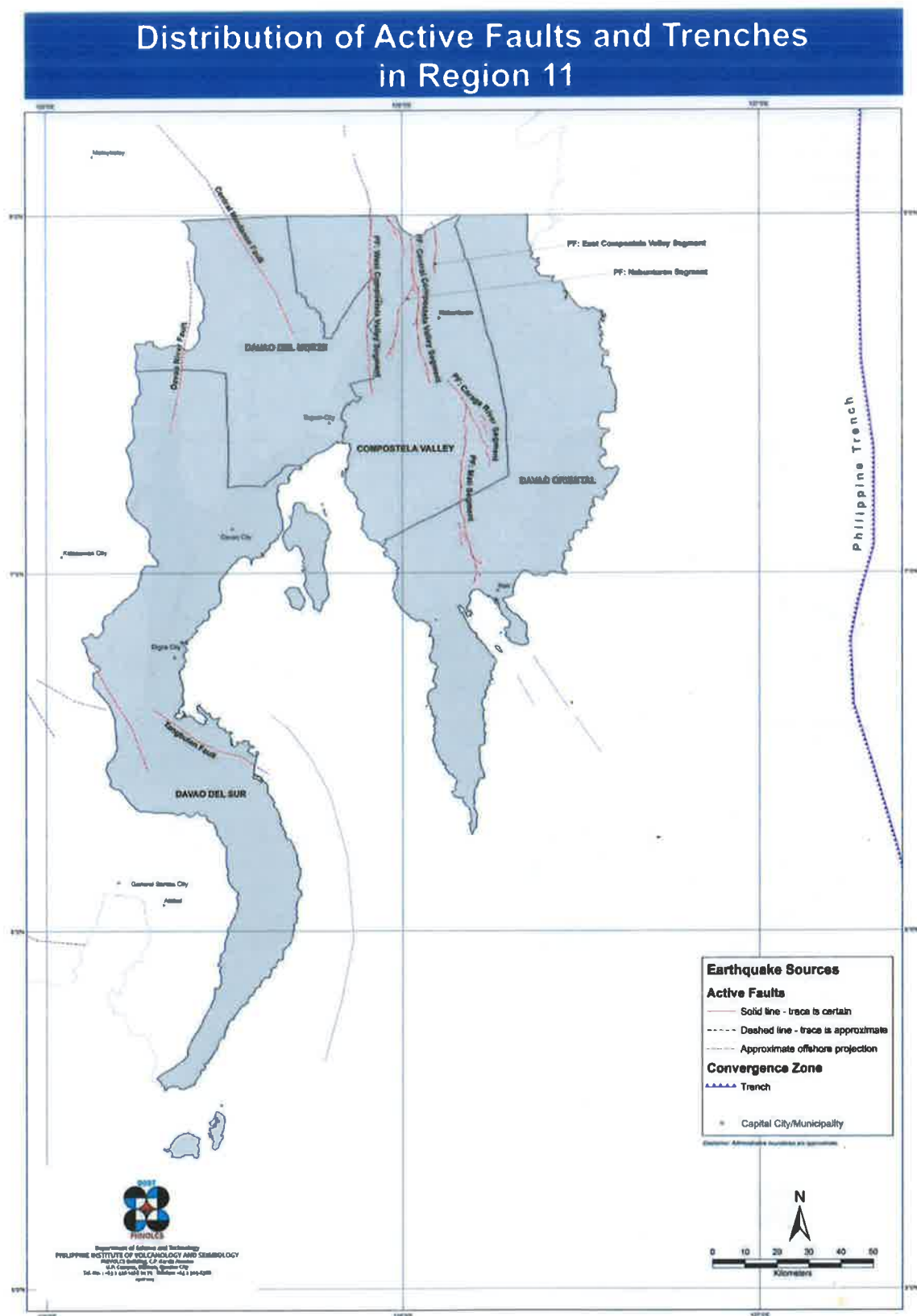


Figure 208-2L Distribution of Active Faults in Region 11

Distribution of Active Faults and Trenches in Region 12

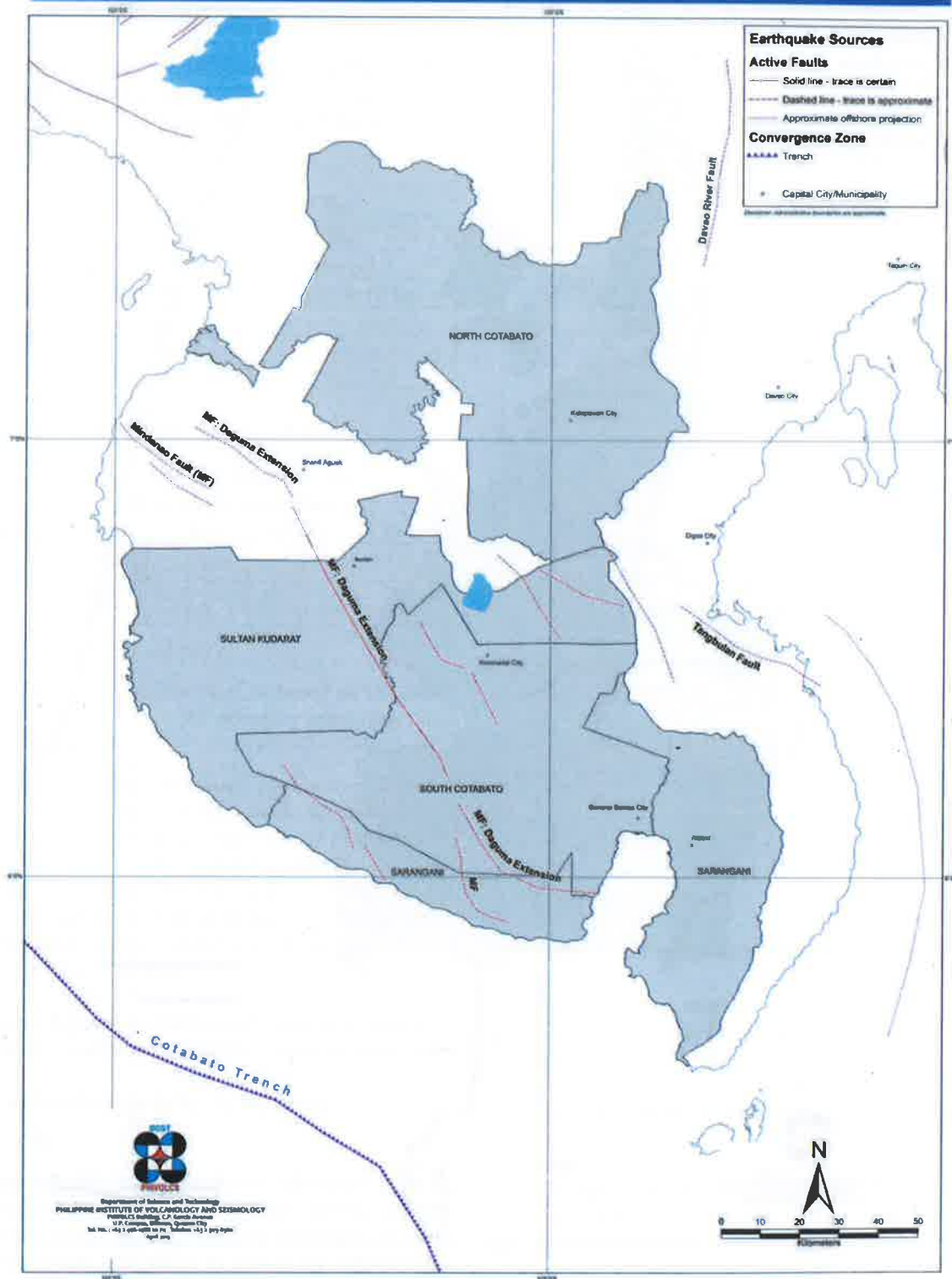


Figure 208-2M Distribution of Active Faults and Trenches in Region 12
 National Structural Code of the Philippines Volume I, 7th Edition, 2015

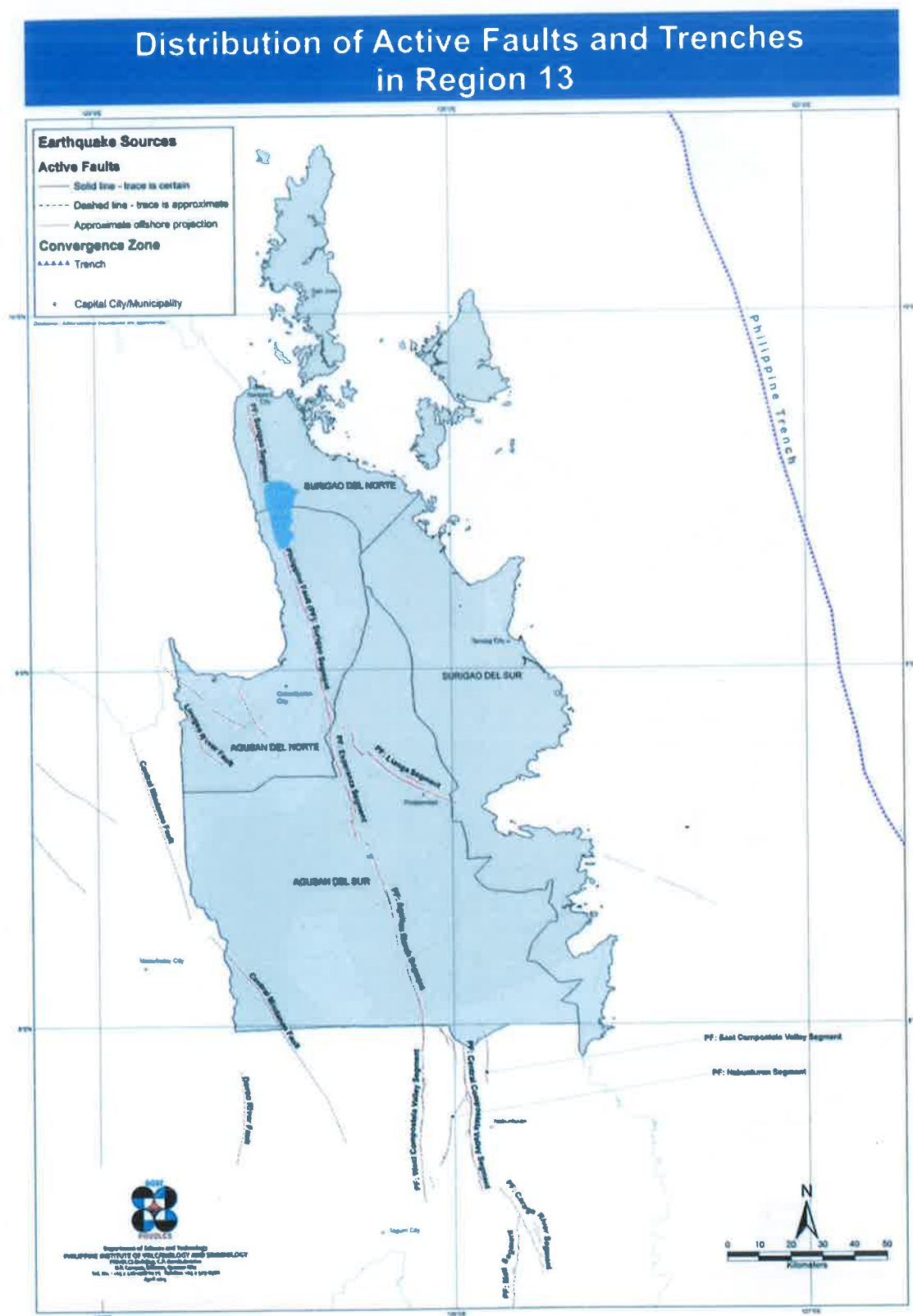


Figure 208-2N Distribution of Active Faults and Trenches in Region 13

208.4.4.3 Seismic Zone 4 Near-Source Factor

In Seismic Zone 4, each site shall be assigned near-source factors in accordance with Tables 208-5 and 208-6 based on the Seismic Source Type as set forth in Section 208.4.4.2.

For high rise structures and essential facilities within 2.0 km of a major fault, a site specific seismic elastic design response spectrum is recommended to be obtained for the specific area.

Table 208-5 Near-Source Factor N_a ¹

Seismic Source Type	Closest Distance To Known Seismic Source ²		
	≤ 2 km	≤ 5 km	≥ 10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

Table 208-6 Near-Source Factor, N_v ¹

Seismic Source Type	Closest Distance To Known Seismic Source ²			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

Notes for Tables 208.5 and 208.6:

- ¹ The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.
- ² The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

The value of N_a used to determine C_a need not exceed 1.1 for structures complying with all the following conditions:

1. The soil profile type is S_A , S_B , S_C or S_D .
2. $\rho = 1.0$.
3. Except in single-storey structures, residential building accommodating 10 or fewer persons, private garages, carports, sheds and agricultural buildings, moment frame systems designated as part of the lateral-force-resisting system shall be special moment-resisting frames.

4. The exceptions to Section 515.6.5 shall not apply, except for columns in one-storey buildings or columns at the top storey of multistorey buildings.
5. None of the following structural irregularities is present: Type 1, 4 or 5 of Table 208-9, and Type 1 or 4 of Table 208-10.

208.4.4.4 Seismic Response Coefficients

Each structure shall be assigned a seismic coefficient, C_a , in accordance with Table 208-7 and a seismic coefficient, C_v , in accordance with Table 208-8.

Table 208-7 Seismic Coefficient, C_a

Soil Profile Type	Seismic Zone Z	
	Z = 0.2	Z = 0.4
S_A	0.16	$0.32N_a$
S_B	0.20	$0.40N_a$
S_C	0.24	$0.40N_a$
S_D	0.28	$0.44N_a$
S_E	0.34	$0.44N_a$
S_F	See Footnote 1 of Table 208-8	

Table 208-8 Seismic Coefficient, C_v

Soil Profile Type	Seismic Zone Z	
	Z = 0.2	Z = 0.4
S_A	0.16	$0.32N_v$
S_B	0.20	$0.40N_v$
S_C	0.32	$0.56N_v$
S_D	0.40	$0.64N_v$
S_E	0.64	$0.96N_v$
S_F	See Footnote 1 of Table 208-8	

- ¹ Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients

208.4.5 Configuration Requirements

Each structure shall be designated as being structurally regular or irregular in accordance with Sections 208.4.5.1 and 208.4.5.2.

208.4.5.1 Regular Structures

Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral-force-resisting systems such as the irregular features described in Section 208.4.5.2.

208.4.5.2 Irregular Structures

- 1. Irregular structures have significant physical discontinuities in configuration or in their lateral-force-resisting systems. Irregular features include, but are not limited to, those described in Tables 208-9 and 208-10. All structures in occupancy Categories 4 and 5 in Seismic Zone 2 need to be evaluated only for vertical irregularities of Type 5 (Table 208-9) and horizontal irregularities of Type 1 (Table 208-10).
- 2. Structures having any of the features listed in Table 208-9 shall be designated as if having a vertical irregularity.

Exception:
Where no storey drift ratio under design lateral forces is greater than 1.3 times the storey drift ratio of the storey above, the structure may be deemed to not have the structural irregularities of Type 1 or 2 in Table 208-9. The storey drift ratio for the top two stories need not be considered. The storey drifts for this determination may be calculated neglecting torsional effects.

- 3. Structures having any of the features listed in Table 208-10 shall be designated as having a plan irregularity.

Table 208-9 Vertical Structural Irregularities

Irregularity Type and Definition	Reference Section
1. Stiffness Irregularity – Soft Storey A soft storey is one in which the lateral stiffness is less than 70 % of that in the storey above or less than 80 percent of the average stiffness of the three stories above.	208.4.8.3 Item 2
2. Weight (Mass) Irregularity Mass irregularity shall be considered to exist where the effective mass of any storey is more than 150 % of the effective mass of an adjacent storey. A roof that is lighter than the floor below need not be considered.	208.4.8.3 Item 2
3. Vertical Geometric Irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any storey is more than 130 % of that in an adjacent storey. One-storey penthouses need not be considered.	208.4.8.3 Item 2
4. In-Plane Discontinuity In Vertical Lateral-Force-Resisting Element Irregularity An in-plane offset of the lateral-load-resisting elements greater than the length of those elements.	208.5.8.1.5. 1
5. Discontinuity In Capacity – Weak Storey Irregularity A weak storey is one in which the storey strength is less than 80 % of that in the storey above. The storey strength is the total strength of all seismic-resisting elements sharing the storey for the direction under consideration.	208.4.9.1

Table 208-10 Horizontal Structural Irregularities

Irregularity Type and Definition	Reference Section
1. Torsional Irregularity - To Be Considered When Diaphragms Are Not Flexible Torsional irregularity shall be considered to exist when the maximum storey drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.	208.7.2.7 Item 6
2. Re-Entrant Corner Irregularity Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 % of the plan dimension of the structure in the given direction.	208.7.2.7 Items 6 and 7
3. Diaphragm Discontinuity Irregularity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 % of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 % from one storey to the next.	208.7.2.7 Item 6
4. Out-Of-Plane Offsets Irregularity Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements	208.5.8.5.1 208.7.2.7 Item 6;
5. Non-parallel Systems Irregularity The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting systems.	208.7.1

208.4.6 Structural Systems

Structural systems shall be classified as one of the types listed in Table 208-11 and defined in this section.

208.4.6.1 Bearing Wall System

A structural system without a complete vertical load-carrying space frame. Bearing walls or bracing systems provide support for all or most gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

208.4.6.2 Building Frame System

A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

208.4.6.3 Moment-Resisting Frame System

A structural system with an essentially complete space frame providing support for gravity loads. Moment-resisting frames provide resistance to lateral load primarily by flexural action of members.

208.4.6.4 Dual System

A structural system with the following features:

1. An essentially complete space frame that provides support for gravity loads.
2. Resistance to lateral load is provided by shear walls or braced frames and moment-resisting frames (SMRF, IMRF, MMRWF or steel OMRF). The moment-resisting frames shall be designed to independently resist at least 25 percent of the design base shear.
3. The two systems shall be designed to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

208.4.6.5 Cantilevered Column System

A structural system relying on cantilevered column elements for lateral resistance.

208.4.6.6 Undefined Structural System

A structural system not listed in Table 208-11.

208.4.6.7 Non-building Structural System

A structural system conforming to Section 208.8.

208.4.7 Height Limits

Height limits for the various structural systems in Seismic Zone 4 are given in Table 208-11.

Exception:
Regular structures may exceed these limits by not more than 50 percent for unoccupied structures, which are not accessible to the general public.

208.4.8 Selection of Lateral Force Procedure

Any structure may be, and certain structures defined below shall be, designed using the dynamic lateral-force procedures of Section 208.5.3.

208.4.8.1 Simplified Static

The simplified static lateral-force procedure set forth in Section 208.5.1.1 may be used for the following structures of Occupancy Category IV or V:

1. Buildings of any occupancy (including single-family dwellings) not more than three stories in height excluding basements that use light-frame construction.
2. Other buildings not more than two stories in height excluding basements.

208.4.8.2 Static

The static lateral force procedure of Section 208.5 may be used for the following structures:

1. All structures, regular or irregular in Occupancy Categories IV and V in Seismic Zone 2.
2. Regular structures under 75 m in height with lateral force resistance provided by systems listed in Table 208-11, except where Section 208.4.8.3, Item 4, applies.
3. Irregular structures not more than five stories or 20 m in height.
4. Structures having a flexible upper portion supported on a rigid lower portion where both portions of the structure considered separately can be classified as being regular, the average storey stiffness of the lower portion is at least 10 times the average storey stiffness of the upper portion and the period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

208.4.8.3 Dynamic

The dynamic lateral-force procedure of Section 208.5.3 shall be used for all other structures, including the following:

1. Structures 75 m or more in height, except as permitted by Section 208.4.8.2, Item 1.
2. Structures having a stiffness, weight or geometric vertical irregularity of Type 1, 2 or 3, as defined in Table 208-9, or structures having irregular features not described in Table 208-9 or 208-10, except as permitted by Section 208.4.10.3.1.
3. Structures over five stories or 20 m in height in Seismic Zone 4 not having the same structural system throughout their height except as permitted by Section 208.5.3.2.
4. Structures, regular or irregular, located on Soil Profile Type S_F , that have a period greater than 0.7 s. The analysis shall include the effects of the soils at the site and shall conform to Section 208.5.3.2, Item 4.

208.4.8.4 Alternative Procedures

208.4.8.4.1 General

Alternative lateral-force procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these provisions.

208.4.8.4.2 Seismic Isolation

Seismic isolation, energy dissipation and damping systems may be used in the analysis and design of structures when approved by the building official and when special detailing is used to provide results equivalent to those obtained by the use of conventional structural systems.

208.4.9 System Limitations

208.4.9.1 Discontinuity

Structures with a discontinuity in capacity, vertical irregularity Type 5 as defined in Table 208-9, shall not be over two stories or 9 m in height where the weak storey has a calculated strength of less than 65 % of the storey above.

Exception:

Where the weak storey is capable of resisting a total lateral seismic force of Ω_o times the design force prescribed in Section 208.5.

208.4.9.2 Undefined Structural Systems

For undefined structural systems not listed in Table 208-11, the coefficient R shall be substantiated by approved cyclic test data and analyses. The following items shall be addressed when establishing R :

1. Dynamic response characteristics,
2. Lateral force resistance,
3. Over-strength and strain hardening or softening,
4. Strength and stiffness degradation,
5. Energy dissipation characteristics,
6. System ductility, and
7. Redundancy.

208.4.9.3 Irregular Features

All structures having irregular features described in Table 208-9 or 208-10 shall be designed to meet the additional requirements of those sections referenced in the tables.

208.4.10 Determination of Seismic Factors

208.4.10.1 Determination of Ω_o

For specific elements of the structure, as specifically identified in this code, the minimum design strength shall be the product of the seismic force over-strength factor Ω_o and the design seismic forces set forth in Section 208.5. For both Allowable Stress Design and Strength Design, the Seismic Force Over-strength Factor, Ω_o , shall be taken from Table 208-11.

208.4.10.2 Determination of R

The value for R shall be taken from Table 208-11.

208.4.10.3 Combinations of Structural Systems

Where combinations of structural systems are incorporated into the same structure, the requirements of this section shall be satisfied.

208.4.10.3.1 Vertical Combinations

The value of R used in the design of any storey shall be less than or equal to the value of R used in the given direction for the storey above.

Exception:

This requirement need not be applied to a storey where the dead weight above that storey is less than 10 percent of the total dead weight of the structure.

Structures may be designed using the procedures of this section under the following conditions:

The entire structure is designed using the lowest R of the lateral force-resisting systems used, or

1. The following two-stage static analysis procedures may be used for structures conforming to Section 208.4.8.2, Item 4.
 - 1.1 The flexible upper portion shall be designed as a separate structure, supported laterally by the rigid lower portion, using the appropriate values of R and ρ .
 - 1.2 The rigid lower portion shall be designed as a separate structure using the appropriate values of R and ρ . The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the (R/ρ) of the upper portion over (R/ρ) of the lower portion.

208.4.10.3.2 Combinations along Different Axes

In Seismic Zone 4 where a structure has a bearing wall system in only one direction, the value of R used for design in the orthogonal direction shall not be greater than that used for the bearing wall system.

Any combination of bearing wall systems, building frame systems, dual systems or moment-resisting frame systems may be used to resist seismic forces in structures less than 50 m in height. Only combinations of dual systems and special moment-resisting frames shall be used to resist

seismic forces in structures exceeding 50 m in height in Seismic Zone 4.

208.4.10.3.3 Combinations along the Same Axis

Where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of R used for design in that direction shall not be greater than the least value for any of the systems utilized in that same direction.

208.5 Minimum Design Lateral Forces and Related Effects**208.5.1 Simplified Static Force Procedure**

Structures conforming to the requirements of Section 208.4.8.1 may be designed using this procedure.

208.5.1.1 Simplified Design Base Shear

The total design base shear in a given direction shall be determined from the following equation:

$$V = \frac{3C_a}{R} W \quad (208-5)$$

where the value of C_a shall be based on Table 208-7 for the soil profile type. When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used in Seismic Zone 4, and Type S_E shall be used in Seismic Zone 2. In Seismic Zone 4, the Near-Source Factor, N_a , need not be greater than 1.2 if none of the following structural irregularities are present:

1. Type 1, 4 or 5 of Table 208-9, or
2. Type 1 or 4 of Table 208-10.

208.5.1.2 Vertical Distribution

The forces at each level shall be calculated using the following equation:

$$F_x = \frac{3C_a}{R} w_i \quad (208-6)$$

where the value of C_a shall be determined as in Section 208.5.1.1.

208.5.1.3 Horizontal Distribution of Shear

The design storey shear, V_x , in any storey is the sum of the forces F_t and F_x above that storey. V_x shall be distributed to the various elements of the vertical lateral force-resisting system in proportion to their rigidities, considering the rigidity of the diaphragm. See Section 208.7.2.3 for rigid elements that are not intended to be part of the lateral force-resisting systems.

Where diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the storey shear distribution shall be considered.

Diaphragms shall be considered flexible for the purposes of distribution of storey shear and torsional moment when the maximum lateral deformation of the diaphragm is more than two times the average storey drift of the associated storey. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral load with the storey drift of adjoining vertical-resisting elements under equivalent tributary lateral load.

208.5.1.4 Horizontal Torsional Moments

Provisions shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered for design.

The torsional design moment at a given storey shall be the moment resulting from eccentricities between applied design lateral forces at levels above that storey and the vertical-resisting elements in that storey plus an accidental torsion.

The accidental torsional moment shall be determined by assuming the mass is displaced as required by Section 208.5.1.3.

Where torsional irregularity exists, as defined in Table 208-10, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x , determined from the following equation:

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}} \right]^2 \quad (208-7)$$

where

- δ_{avg} = the average of the displacements at the extreme points of the structure at Level x , mm
- δ_{max} = the maximum displacement at Level x , mm

The value of A_x need not exceed 3.0

208.5.1.5 Overturning

Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in Section 208.5.2.3. At any level, the overturning moments to be resisted shall be determined using those seismic forces (F_t and F_x) that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in Section 208.5.1.3. Overturning effects on every element shall be carried down to the foundation. See Sections 207.1 and 208.7 for combining gravity and seismic forces.

208.5.1.5.1 Elements Supporting Discontinuous Systems

208.5.1.5.1.1 General

Where any portion of the lateral load-resisting system is discontinuous, such as for vertical irregularity Type 4 in Table 208-9 or plan irregularity Type 4 in Table 208-10, concrete, masonry, steel and wood elements supporting such discontinuous systems shall have the design strength to resist the combination loads resulting from the special seismic load combinations of Section 203.5.

Exceptions:

1. The quantity E_m in Section 208.6 need not exceed the maximum force that can be transferred to the element by the lateral-force-resisting system.
2. Concrete slabs supporting light-frame wood shear wall systems or light-frame steel and wood structural panel shear wall systems.

For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ , of 1.0. This increase shall not be combined with the one-third stress increase permitted by Section 203.4, but may be combined with the duration of load increase permitted in Section 615.3.4.

208.5.1.5.1.1.2 Detailing requirements in Seismic Zone 4

In Seismic Zone 4, elements supporting discontinuous systems shall meet the following detailing or member limitations:

1. Reinforced concrete or reinforced masonry elements designed primarily as axial-load members shall comply with Section 421.4.4.5.
2. Reinforced concrete elements designed primarily as flexural members and supporting other than light-frame wood shear wall system or light-frame steel and wood structural panel shear wall systems shall comply with Sections 421.3.2 and 421.3.3. Strength computations for portions of slabs designed as supporting elements shall include only those portions of the slab that comply with the requirements of these sections.
3. Masonry elements designed primarily as axial-load carrying members shall comply with Sections 706.1.12.4, Item 1, and 708.2.6.2.6.
4. Masonry elements designed primarily as flexural members shall comply with Section 708.2.6.2.5.
5. Steel elements designed primarily as axial-load members shall comply with Sections 515.4.2 and 515.4.3.
6. Steel elements designed primarily as flexural members or trusses shall have bracing for both top and bottom beam flanges or chords at the location of the support of the discontinuous system and shall comply with the requirements of Section 515.6.1.3.
7. Wood elements designed primarily as flexural members shall be provided with lateral bracing or solid blocking at each end of the element and at the connection location(s) of the discontinuous system.

208.5.1.5.2 At Foundation

See Sections 208.4.1 and 308.4 for overturning moments to be resisted at the foundation soil interface.

208.5.1.6 Applicability

Sections 208.6.2, 208.6.3, 208.5.2.1, 208.5.2.2, 208.5.2.3, 208.6.4, 208.6.5 and 208.5.3 shall not apply when using the simplified procedure.

Exception:

For buildings with relatively flexible structural systems, the building official may require consideration of $P\Delta$ effects and drift in accordance with Sections 208.6.3, 208.6.4 and 208.6.5. Δ_s shall be determined using design seismic forces from Section 208.5.1.1.

Where used, Δ_M shall be taken equal to 0.01 times the storey height of all stories. In Section 208.7.2.7, Equation 208-22 shall read $F_{px} = \frac{3C_a}{R} w_{px}$ and need not exceed $C_a w_{px}$, but shall not be less than $0.5C_a w_{px}$. R and Ω_o shall be taken from Table 208-11.

208.5.2 Static Force Procedure**208.5.2.1 Design Base Shear**

The total design base shear in a given direction shall be determined from the following equation:

$$V = \frac{C_v I}{RT} W \quad (208-8)$$

The total design base shear need not exceed the following:

$$V = \frac{2.5C_a I}{R} W \quad (208-9)$$

The total design base shear shall not be less than the following:

$$V = 0.11C_a I W \quad (208-10)$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8ZN_v I}{R} W \quad (208-11)$$

208.5.2.2 Structure Period

The value of T shall be determined from one of the following methods:

1. Method A:

For all buildings, the value T may be approximated from the following equation:

$$T = C_t(h_n)^{3/4} \quad (208-12)$$

where

$$\begin{aligned} C_t &= 0.0853 \text{ for steel moment-resisting frames} \\ C_t &= 0.0731 \text{ for reinforced concrete moment-resisting frames and eccentrically braced frames} \\ C_t &= 0.0488 \text{ for all other buildings} \end{aligned}$$

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $0.0743/\sqrt{A_c}$.

The value of A_c shall be determined from the following equation:

$$A_c = \sum A_e[0.2 + (D_e/h_n)^2] \quad (208-13)$$

The value of D_e/h_n used in Equation 208-13 shall not exceed 0.9.

2. Method B:

The fundamental period T may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The analysis shall be in accordance with the requirements of Section 208.6.2. The value of T from Method B shall not exceed a value 30 percent greater than the value of T obtained from Method A in Seismic Zone 4, and 40 percent in Seismic Zone 2.

The fundamental period T may be computed by using the following equation:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^n w_i \delta_i^2}{g \sum_{i=1}^n w_i f_i \delta_i}} \quad (208-14)$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Equations 208-15, 208-16 and 208-17 or any other rational

distribution. The elastic deflections, δ_i , shall be calculated using the applied lateral forces, f_i .

208.5.2.3 Vertical Distribution of Force

The total force shall be distributed over the height of the structure in conformance with Equations 208-15, 208-16 and 208-17 in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^n F_i \quad (208-15)$$

The concentrated force F_t at the top, which is in addition to F_n , shall be determined from the equation:

$$F_t = 0.07TV \quad (208-16)$$

The value of T used for the purpose of calculating F_t shall be the period that corresponds with the design base shear as computed using Equation 208-4. F_t need not exceed $0.25V$ and may be considered as zero where T is 0.7 s or less. The remaining portion of the base shear shall be distributed over the height of the structure, including Level n , according to the following equation:

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^n w_i h_i} \quad (208-17)$$

At each level designated as x , the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. Structural displacements and design seismic forces shall be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

208.5.3 Dynamic Analysis Procedures

208.5.3.1 General

Dynamic analyses procedures, when used, shall conform to the criteria established in this section. The analysis shall be based on an appropriate ground motion representation and shall be performed using accepted principles of dynamics.

Structures that are designed in accordance with this section shall comply with all other applicable requirements of these provisions.

208.5.3.2 Ground Motion

The ground motion representation shall, as a minimum, be one having a 10-percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

1. An elastic design response spectrum constructed in accordance with Figure 208-3, using the values of C_a and C_v consistent with the specific site. The design acceleration ordinates shall be multiplied by the acceleration of gravity, 9.815 m/sec^2 .
2. A site-specific elastic design response spectrum based on the geologic, tectonic, seismologic and soil characteristics associated with the specific site. The spectrum shall be developed for a damping ratio of 0.05, unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site.
3. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site design spectrum conforming to Section 208.5.3.2, Item 2.

4. For structures on Soil Profile Type S_F , the following requirements shall apply when required by Section 208.4.8.3, Item 4:

4.1 The ground motion representation shall be developed in accordance with Items 2 and 3.

4.2 Possible amplification of building response due to the effects of soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.

5. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two-thirds. Alternative factors may be used when substantiated by site-specific data. Where the Near-Source Factor, N_a , is greater than 1.0, site-specific vertical response spectra shall be used in lieu of the factor of two-thirds.

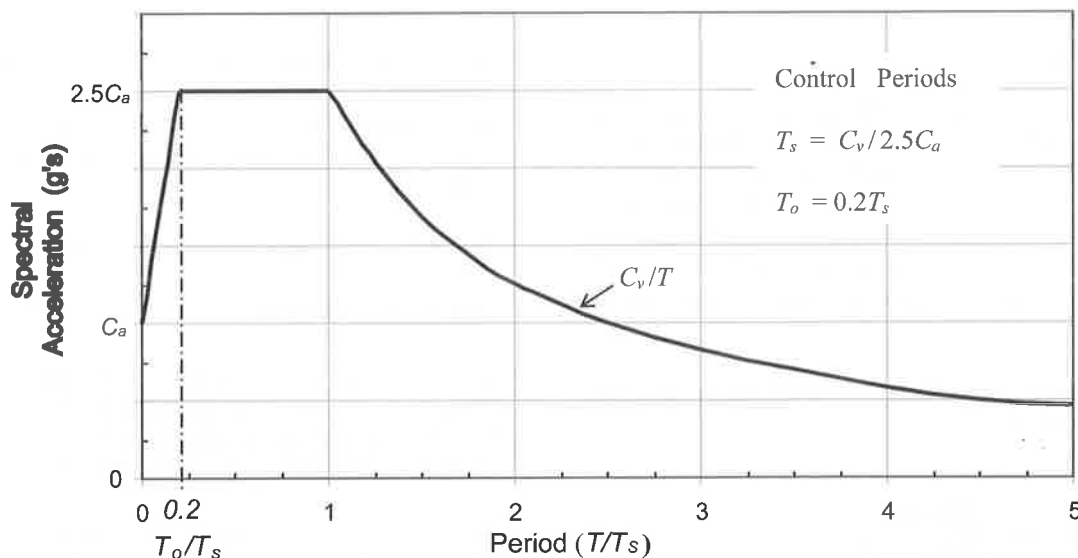


Figure 208-3
Design Response Spectra

208.5.3.3 Mathematical Model

A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response. A three-dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those having a plan irregularity defined in Table 208-10 and having a rigid or semi-rigid diaphragm. The stiffness properties used in the analysis and general mathematical modeling shall be in accordance with Section 208.6.2.

208.5.3.4 Description of Analysis Procedures

208.5.3.4.1 Response Spectrum Analysis

An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

208.5.3.4.2 Time History Analysis

An analysis of the dynamic response of a structure at each increment of time when the base is subjected to a specific ground motion time history.

208.5.3.5 Response Spectrum Analysis

208.5.3.5.1 Response Spectrum Representation and Interpretation of Results

The ground motion representation shall be in accordance with Section 208.5.3.2. The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters. Elastic Response Parameters may be reduced in accordance with Section 208.5.3.5.4.

The base shear for a given direction, determined using dynamic analysis must not be less than the value obtained by the equivalent lateral force method of Section 208.5.2. In this case, all corresponding response parameters are adjusted proportionately.

208.5.3.5.2 Number of Modes

The requirement of Section 208.5.3.4.1 that all significant modes be included may be satisfied by demonstrating that

for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

208.5.3.5.3 Combining Modes

The peak member forces, displacements, storey forces, storey shears and base reactions for each mode shall be combined by recognized methods. When three-dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

208.5.3.5.4 Reduction of Elastic Response Parameters for Design

Elastic Response Parameters may be reduced for purposes of design in accordance with the following items, with the limitation that in no case shall the Elastic Response Parameters be reduced such that the corresponding design base shear is less than the Elastic Response Base Shear divided by the value of R .

1. For all regular structures where the ground motion representation complies with Section 208.5.3.2, Item 1, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 90 percent of the base shear determined in accordance with Section 208.5.2.
2. For all regular structures where the ground motion representation complies with Section 208.5.3.2, Item 2, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 80 percent of the base shear determined in accordance with Section 208.5.2.
3. For all irregular structures, regardless of the ground motion representation, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 100 percent of the base shear determined in accordance with Section 208.5.2.

The corresponding reduced design seismic forces shall be used for design in accordance with Section 203.

208.5.3.5.5 Directional Effects

Directional effects for horizontal ground motion shall conform to the requirements of Section 208.6. The effects of vertical ground motions on horizontal cantilevers and pre-stressed elements shall be considered in accordance with Section 208.6. Alternately, vertical seismic response may be determined by dynamic response methods; in no

case shall the response used for design be less than that obtained by the static method.

208.5.3.5.6 Torsion

The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 208.5.1.4. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent static procedures such as provided in Section 208.5.1.3.

208.5.3.5.7 Dual Systems

Where the lateral forces are resisted by a dual system as defined in Section 208.4.6.4, the combined system shall be capable of resisting the base shear determined in accordance with this section. The moment-resisting frame shall conform to Section 208.4.6.4, Item 2, and may be analyzed using either the procedures of Section 208.5.2.3 or those of Section 208.5.3.5.

208.5.3.6 Time History Analysis

208.5.3.6.1 Time History

Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time-history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design-basis earthquake (or maximum capable earthquake). Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs may be used to make up the total number required. For each pair of horizontal ground-motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site-specific spectrum of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake for periods from **0.2T** second to **1.5T** seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects.

The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

208.5.3.6.2 Elastic Time History Analysis

Elastic time history shall conform to Sections 208.5.3.1, 208.5.3.2, 208.5.3.3, 208.5.3.5.2, 208.5.3.5.4, 208.6.5.3.5.5, 208.6.5.3.5.6, 208.5.3.5.7 and 208.5.3.6.1 and 208.6.6.1. Response parameters from elastic time-history analysis shall be denoted as Elastic Response Parameters. All elements shall be designed using Strength Design. Elastic Response Parameters may be scaled in accordance with Section 208.5.3.5.4.

208.5.3.6.3 Nonlinear Time History Analysis

208.5.3.6.3.1 Nonlinear Time History

Nonlinear time history analysis shall meet the requirements of Section 208.4.8.4, and time histories shall be developed and results determined in accordance with the requirements of Section 208.5.3.6.1. Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, considering the Importance Factor. The maximum inelastic response displacement shall not be reduced and shall comply with Section 208.6.5.

208.5.3.6.3.2 Design Review

When nonlinear time-history analysis is used to justify a structural design, a design review of the lateral-force-resisting system shall be performed by an independent engineering team, including persons licensed in the appropriate disciplines and experienced in seismic analysis methods. The lateral-force-resisting system design review shall include, but not be limited to, the following:

1. Reviewing the development of site-specific spectra and ground-motion time histories.
2. Reviewing the preliminary design of the lateral-force-resisting system.
3. Reviewing the final design of the lateral-force-resisting system and all supporting analyses.

The engineer-of-record shall submit with the plans and calculations a statement by all members of the engineering team doing the review stating that the above review has been performed.

208.6 Earthquake Loads and Modeling Requirements

208.6.1 Earthquake Loads

Structures shall be designed for ground motion producing structural response and seismic forces in any horizontal direction. The following earthquake loads shall be used in the load combinations set forth in Section 203:

$$E = \rho E_h + E_v \quad (208-18)$$

$$E_m = \Omega_o E_h \quad (208-19)$$

where

- E = the earthquake load on an element of the structure resulting from the combination of the horizontal component, E_h , and the vertical component, E_v
- E_h = the earthquake load due to the base shear, V , as set forth in Section 208.5.2 or the design lateral force, F_p , as set forth in Section 208.9
- E_m = the estimated maximum earthquake force that can be developed in the structure as set forth in Section 208.6.1, and used in the design of specific elements of the structure, as specifically identified in this section
- E_v = the load effect resulting from the vertical component of the earthquake ground motion and is equal to an addition of $0.5C_aID$ to the dead load effect, D , for Strength Design, and may be taken as zero for Allowable Stress Design
- Ω_o = the seismic force amplification factor that is required to account for structural overstrength, as set forth in Section 208.4.10.1
- ρ = Reliability/Redundancy Factor as given by the following equation:

$$\rho = 2 - \frac{6.1}{r_{max}\sqrt{A_B}} \quad (208-20)$$

where

- r_{max} = the maximum element-storey shear ratio. For a given direction of loading, the element-storey shear ratio is the ratio of the design storey shear in the most heavily loaded single element divided by the total design storey shear.

For any given Storey Level i , the element-storey shear ratio is denoted as r_i . The maximum element-storey shear ratio

r_{max} is defined as the largest of the element storey shear ratios, r_i , which occurs in any of the storey levels at or below the two-thirds height level of the building.

For braced frames, the value of r_i is equal to the maximum horizontal force component in a single brace element divided by the total storey shear.

For moment frames, r_i shall be taken as the maximum of the sum of the shears in any two adjacent columns in a moment frame bay divided by the storey shear. For columns common to two bays with moment-resisting connections on opposite sides at Level i in the direction under consideration, 70 percent of the shear in that column may be used in the column shear summation.

For shear walls, r_i shall be taken as the maximum value of the product of the wall shear multiplied by $3/l_w$ and divided by the total storey shear, where l_w is the length of the wall in meter.

For dual systems, r_i shall be taken as the maximum value of r_i as defined above considering all lateral-load-resisting elements. The lateral loads shall be distributed to elements based on relative rigidities considering the interaction of the dual system. For dual systems, the value of ρ need not exceed 80 percent of the value calculated above.

ρ shall not be taken less than 1.0 and need not be greater than 1.5. For special moment-resisting frames, except when used in dual systems, ρ shall not exceed 1.25. The number of bays of special moment-resisting frames shall be increased to reduce r , such that ρ is less than or equal to 1.25.

Exception:

A_B may be taken as the average floor area in the upper setback portion of the building where a larger base area exists at the ground floor.

When calculating drift, or when the structure is located in Seismic Zone 2, ρ shall be taken equal to 1.0.

The ground motion producing lateral response and design seismic forces may be assumed to act non-concurrently in the direction of each principal axis of the structure, except as required by Section 208.7.2.

Seismic dead load, W , is the total dead load and applicable portions of other loads listed below.

1. In storage and warehouse occupancies, a minimum of 25 percent of the floor live load shall be applicable.

2. Where a partition load is used in the floor design, a load of not less than 0.5 kN/m^2 shall be included.
3. Total weight of permanent equipment shall be included.

208.6.2 Modeling Requirements

The mathematical model of the physical structure shall include all elements of the lateral-force-resisting system. The model shall also include the stiffness and strength of elements, which are significant to the distribution of forces, and shall represent the spatial distribution of the mass and stiffness of the structure. In addition, the model shall comply with the following:

1. Stiffness properties of reinforced concrete and masonry elements shall consider the effects of cracked sections.
2. For steel moment frame systems, the contribution of panel zone deformations to overall storey drift shall be included.

208.6.3 $P\Delta$ Effects

The resulting member forces and moments and the storey drifts induced by $P\Delta$ effects shall be considered in the evaluation of overall structural frame stability and shall be evaluated using the forces producing the displacements of Δ_s . $P\Delta$ need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10; the ratio may be evaluated for any storey as the product of the total dead and floor live loads, as required in Section 203, above the storey times the seismic drift in that storey divided by the product of the seismic shear in that storey times the height of that storey. In Seismic Zone 4, $P\Delta$ need not be considered when the storey drift ratio does not exceed $0.02/R$.

208.6.4 Drift

Drift or horizontal displacements of the structure shall be computed where required by this code. For both Allowable Stress Design and Strength Design, the Maximum Inelastic Response Displacement, Δ_M , of the structure caused by the Design Basis Ground Motion shall be determined in accordance with this section. The drifts corresponding to the design seismic forces of Section 208.5.2.1 or Section 208.5.3.5, Δ_s , shall be determined in accordance with Section 208.6.4.1. To determine Δ_M , these drifts shall be amplified in accordance with Section 208.6.4.2.

208.6.4.1 Determination of Δ_s

A static, elastic analysis of the lateral force-resisting system shall be prepared using the design seismic forces from Section 208.5.2.1. Alternatively, dynamic analysis may be performed in accordance with Section 208.5.3. Where Allowable Stress Design is used and where drift is being computed, the load combinations of Section 203.3 shall be used. The mathematical model shall comply with Section 208.6.2. The resulting deformations, denoted as Δ_s , shall be determined at all critical locations in the structure. Calculated drift shall include translational and torsional deflections.

208.6.4.2 Determination of Δ_M

The Maximum Inelastic Response Displacement, Δ_M , shall be computed as follows:

$$\Delta_M = 0.7R\Delta_s \quad (208-21)$$

Exception:

Alternatively, Δ_m may be computed by nonlinear time history analysis in accordance with Section 208.5.3.6.3.

The analysis used to determine the Maximum Inelastic Response Displacement Δ_M shall consider $P\Delta$ effects.

208.6.5 Storey Drift Limitation

Storey drifts shall be computed using the Maximum Inelastic Response Displacement, Δ_M .

208.6.5.1 Calculated

Calculated storey drift using Δ_M shall not exceed 0.025 times the storey height for structures having a fundamental period of less than 0.7 sec. For structures having a fundamental period of 0.7 sec or greater, the calculated storey drift shall not exceed 0.020 times the storey height.

Exceptions:

1. *These drift limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety. The drift used in this assessment shall be based upon the Maximum Inelastic Response Displacement, Δ_M .*
2. *There shall be no drift limit in single-storey steel-framed structures whose primary use is limited to storage, factories or workshops. Minor accessory uses shall be allowed. Structures on which this exception is used shall not have equipment attached to*

the structural frame or shall have such equipment detailed to accommodate the additional drift. Walls that are laterally supported by the steel frame shall be designed to accommodate the drift in accordance with Section 208.7.2.3.

208.6.5.2 Limitations

The design lateral forces used to determine the calculated drift may disregard the limitations of Equations. 208-11 and 208-10 and may be based on the period determined from Equations. 208-14 neglecting the 30 or 40 percent limitations of Section 208.5.2.2.

208.6.6 Vertical Component

The following requirements apply in Seismic Zone 4 only. Horizontal cantilever components shall be designed for a net upward force of $0.7C_aIW_p$.

In addition to all other applicable load combinations, horizontal pre-stressed components shall be designed using not more than 50 percent of the dead load for the gravity load, alone or in combination with the lateral force effects.

208.7 Detailed Systems Design Requirements

208.7.1 General

All structural framing systems shall comply with the requirements of Section 208.4. Only the elements of the designated seismic-force-resisting system shall be used to resist design forces. The individual components shall be designed to resist the prescribed design seismic forces acting on them. The components shall also comply with the specific requirements for the material contained in Chapters 4 through 7. In addition, such framing systems and components shall comply with the detailed system design requirements contained in Section 208.7.

All building components in Seismic Zones 2 and 4 shall be designed to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead and floor live loads.

Consideration shall be given to design for uplift effects caused by seismic loads.

In Seismic Zones 2 and 4, provision shall be made for the effects of earthquake forces acting in a direction other than the principal axes in each of the following circumstances:

1. The structure has plan irregularity Type 5 as given in Table 208-10.
2. The structure has plan irregularity Type 1 as given in Table 208-10 for both major axes.
3. A column of a structure forms part of two or more intersecting lateral-force-resisting systems.

Exception:

If the axial load in the column due to seismic forces acting in either direction is less than 20 percent of the column axial load capacity.

The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed design seismic forces in one direction plus 30 percent of the prescribed design seismic forces in the perpendicular direction. The combination requiring the greater component strength shall be used for design. Alternatively, the effects of the two orthogonal directions may be combined on a square root of the sum of the squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative result.

8.7.2 Structural Framing Systems

Four types of general building framing systems defined in Section 208.4.6 are recognized in these provisions and shown in Table 208-11. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing requirements are given in this section and in Chapters 4 through 7.

8.7.2.1 Detailing for Combinations of Systems

For components common to different structural systems, the more restrictive detailing requirements shall be used.

8.7.2.2 Connections

Connections that resist design seismic forces shall be designed and detailed on the drawings.

8.7.2.3 Deformation Compatibility

All structural framing elements and their connections, not required by design to be part of the lateral-force-resisting system, shall be designed and/or detailed to be adequate to maintain support of design dead plus live loads when subjected to the expected deformations caused by seismic forces. $P\Delta$ effects on such elements shall be considered. Expected deformations shall be determined as the greater

of the Maximum Inelastic Response Displacement, Δ_M , considering $P\Delta$ effects determined in accordance with Section 208.6.4.2 or the deformation induced by a storey drift of 0.0025 times the storey height. When computing expected deformations, the stiffening effect of those elements not part of the lateral-force-resisting system shall be neglected.

For elements not part of the lateral-force-resisting system, the forces induced by the expected deformation may be considered as ultimate or factored forces. When computing the forces induced by expected deformations, the restraining effect of adjoining rigid structures and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used. Inelastic deformations of members and connections may be considered in the evaluation, provided the assumed calculated capacities are consistent with member and connection design and detailing.

For concrete and masonry elements that are part of the lateral-force-resisting system, the assumed flexural and shear stiffness properties shall not exceed one half of the gross section properties unless a rational cracked-section analysis is performed. Additional deformations that may result from foundation flexibility and diaphragm deflections shall be considered. For concrete elements not part of the lateral-force-resisting system, see Section 421.9.

Table 208-11A Earthquake-Force-Resisting Structural Systems of Concrete

Basic Seismic-Force Resisting System	R	Ω_0	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
A. Bearing Wall Systems				
• Special reinforced concrete shear walls	4.5	2.8	NL	50
• Ordinary reinforced concrete shear walls	4.5	2.8	NL	NP
B. Building Frame Systems				
• Special reinforced concrete shear walls or braced frames (shear walls)	5.0	2.8	NL	75
• Ordinary reinforced concrete shear walls or braced frames	5.6	2.2	NL	NP
• Intermediate precast shear walls or braced frames	5.0	2.5	NL	10
C. Moment-Resisting Frame Systems				
• Special reinforced concrete moment frames	8.5	2.8	NL	NL
• Intermediate reinforced concrete moment frames	5.5	2.8	NL	NP
• Ordinary reinforced concrete moment frames	3.5	2.8	NL	NP
D. Dual Systems				
• Special reinforced concrete shear walls	8.5	2.8	NL	NL
• Ordinary reinforced concrete shear walls	6.5	2.8	NL	NP
E. Dual System with Intermediate Moment Frames				
• Special reinforced concrete shear walls	6.5	2.8	NL	50
• Ordinary reinforced concrete shear walls	5.5	2.8	NL	NP
• Shear wall frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls	4.2	2.8	NP	NP
F. Cantilevered Column Building Systems				
• Cantilevered column elements	2.2	2.0	NL	10
G. Shear Wall- Frame Interaction Systems	5.5	2.8	NL	50

Table 208-11B Earthquake-Force-Resisting Structural Systems of Steel

Basic Seismic-Force Resisting System	R	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
A. Bearing Wall Systems				
• Light steel-framed bearing walls with tension-only bracing	2.8	2.2	NL	20
• Braced frames where bracing carries gravity load	4.4	2.2	NL	50
• Light framed walls sheathed with steel sheets structural panels rated for shear resistance or steel sheets	5.5	2.8	NL	20
• Light-framed walls with shear panels of all other light materials	4.5	2.8	NL	20
• Light-framed wall systems using flat strap bracing	2.8	2.2	NL	NP
B. Building Frame Systems				
• Steel eccentrically braced frames (EBF), moment-resisting connections at columns away from links	8.0	2.8	NL	30
• Steel eccentrically braced frames (EBF), non-moment-resisting connections at columns away from links	6.0	2.2	NL	30
• Special concentrically braced frames (SCBF)	6.0	2.2	NL	30
• Ordinary concentrically braced frames (OCBF)	3.2	2.2	NL	NP
• Light-framed walls sheathed with steel sheet structural panels / sheet steel panels	6.5	2.8	NL	20
• Light frame walls with shear panels of all other materials	2.5	2.8	NL	NP
• Buckling-restrained braced frames (BRBF), non-moment-resisting beam-column connection	7.0	2.8	NL	30
• Buckling-restrained braced frames, moment-resisting beam-column connections	8.0	2.8	NL	30
• Special steel plate shear walls (SPSW)	7.0	2.8	NL	30
C. Moment-Resisting Frame Systems				
• Special moment-resisting frame (SMRF)	8.0	3.0	NL	NL
• Intermediate steel moment frames (IMF)	4.5	3.0	NL	NP
• Ordinary moment frames (OMF)	3.5	3.0	NL	NP
• Special truss moment frames (STMF)	6.5	3.0	NL	NP
• Special composite steel and concrete moment frames	8.0	3.0	NL	NL
• Intermediate composite moment frames	5.0	3.0	NL	NP
• Composite partially restrained moment frames	6.0	3.0	50	NP
• Ordinary composite moment frames	3.0	3.0	NP	NP
D. Dual Systems with Special Moment Frames				
• Steel eccentrically braced frames	8.0	2.8	NL	NL
• Special steel concentrically braced frames	7.0	2.8	NL	NL
• Composite steel and concrete eccentrically braced frame	8.0	2.8	NL	NL

Table 208-11B (continued) Earthquake-Force-Resisting Structural Systems of Steel

Basic Seismic-Force Resisting System	R	Ω_0	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
• Composite steel and concrete concentrically braced frame	6.0	2.8	NL	NL
• Composite steel plate shear walls	7.5	2.8	NL	NL
• Buckling-restrained braced frame	8.0	2.8	NL	NL
• Special steel plate shear walls	8.0	2.8	NL	NL
• Masonry shear wall with steel OMRF	4.2	2.8	NL	50
• Steel EBF with steel SMRF	8.5	2.8	NL	NL
• Steel EBF with steel OMRF	4.2	2.8	NL	50
• Special concentrically braced frames with steel SMRF	7.5	2.8	NL	NL
• Special concentrically braced frames with steel OMRF	4.2	2.8	NL	50
E. Dual System with Intermediate Moment Frames				
• Special steel concentrically braced frame	6.0	2.8	NL	NP
• Composite steel and concrete concentrically braced frame	5.5	2.8	NL	NP
• Ordinary composite braced frame	3.5	2.8	NL	NP
• Ordinary composite reinforced concrete shear walls with steel elements	5.0	3.0	NL	NP
F. Cantilevered Column Building Systems				
• Special steel moment frames	2.2	2.0	10	10
• Intermediate steel moment frames	1.2	2.0	10	NP
• Ordinary steel moment frames	1.0	2.0	10	NP
• Cantilevered column elements	2.2	2.0	NL	10
G. Steel Systems not Specifically Detailed for Seismic Resistance, Excluding Cantilever Systems	3.0	3.0	NL	NP

Table 208-11C Earthquake-Force-Resisting Structural Systems of Masonry

Basic Seismic-Force Resisting System	<i>R</i>	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
A. Bearing Wall Systems				
• Masonry shear walls	4.5	2.8	NL	50
B. Building Frame Systems				
• Masonry shear walls	5.5	2.8	NL	50
C. Moment-Resisting Frame Systems				
• Masonry moment-resisting wall frames (MMRWF)	6.5	2.8	NL	50
D. Dual Systems				
• Masonry shear walls with SMRF	5.5	2.8	NL	50
• Masonry shear walls with steel OMRF	4.2	2.8	NL	50
• Masonry shear walls with concrete IMRF	4.2	2.8	NL	NP
• Masonry shear walls with masonry MMRWF	6.0	2.8	NL	50

Table 208-11D Earthquake-Force-Resisting Structural Systems of Wood

Basic Seismic-Force Resisting System	<i>R</i>	Ω_o	System Limitation and Building Height Limitation by Seismic Zone (meters)	
			Zone 2	Zone 4
A. Bearing Wall Systems				
• Light-framed walls with shear panels: wood structural panel walls for structures three stories or less	5.5	2.8	NL	20
• Heavy timber braced frames where bracing carries gravity load	2.8	2.2	NL	20
• All other light framed walls	NA	NA		
B. Building Frame Systems				
• Ordinary heavy timber-braced frames	5.6	2.2	NL	20

208.7.2.3.1 Adjoining Rigid Elements

Moment-resisting frames and shear walls may be enclosed by or adjoined by more rigid elements; provided it can be shown that the participation or failure of the more rigid elements will not impair the vertical and lateral- load-resisting ability of the gravity load and lateral-force-resisting systems. The effects of adjoining rigid elements shall be considered when assessing whether a structure shall be designated regular or irregular in Section 208.4.5.

208.7.2.3.2 Exterior Elements

Exterior non-bearing, non-shear wall panels or elements that are attached to or enclose the exterior shall be designed to resist the forces per Equation 208-27 or 208-28 and shall accommodate movements of the structure based on Δ_M and temperature changes. Such elements shall be supported by means of cast-in-place concrete or by mechanical

connections and fasteners in accordance with the following provisions:

1. Connections and panel joints shall allow for a relative movement between stories of not less than two times storey drift caused by wind, the calculated storey drift based on Δ_M or 12.7 mm, whichever is greater.
2. Connections to permit movement in the plane of the panel for storey drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.
3. Bodies of connections shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds.

4. The body of the connection shall be designed for the force determined by Equation 208-28, where $R_p = 3.0$ and $a_p = 1.0$.
5. All fasteners in the connecting system, such as bolts, inserts, welds and dowels, shall be designed for the forces determined by Equation 208-28, where $R_p = 1.0$ and $a_p = 1.0$.
6. Fasteners embedded in concrete shall be attached to, or hooked around, reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel.

208.7.2.3.3 Ties and Continuity

All parts of a structure shall be interconnected and the connections shall be capable of transmitting the seismic force induced by the parts being connected. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having at least strength to resist $0.5C_dI$ times the weight of the smaller portion.

A positive connection for resisting horizontal force acting parallel to the member shall be provided for each beam, girder or truss. This force shall not be less than $0.3C_dI$ times the dead plus live load.

208.7.2.4 Collector Elements

Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces.

Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Equation 208-22. In addition, collector elements, splices, and their connections to resisting elements shall have the design strength to resist the combined loads resulting from the special seismic load of Section 203.5.

Exception:

In structures, or portions thereof, braced entirely by light-frame wood shear walls or light-frame steel and wood structural panel shear wall systems, collector elements, splices and connections to resisting elements need only be designed to resist forces in accordance with Equation 208-22.

The quantity E_M need not exceed the maximum force that can be transferred to the collector by the diaphragm and

other elements of the lateral-force-resisting system. For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ , of 1.0. This increase shall not be combined with the one-third stress increase permitted by Section 203.4, but may be combined with the duration of load increase permitted in Section 615.3.4.

208.7.2.5 Concrete Frames

Concrete frames required by design to be part of the lateral-force-resisting system shall conform to the following:

1. In Seismic Zone 4 they shall be special moment-resisting frames.
2. In Seismic Zone 2 they shall, as a minimum, be intermediate moment-resisting frames.

208.7.2.6 Anchorage of Concrete or Masonry Walls

Concrete or masonry walls shall be anchored to all floors and roofs that provide out-of-plane lateral support of the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof construction capable of resisting the larger of the horizontal forces specified in this section and Sections 206.4 and 208.9. In addition, in Seismic Zone 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel. Requirements for developing anchorage forces in diaphragms are given in Section 208.7.2.6. Diaphragm deformation shall be considered in the design of the supported walls.

208.7.2.6.1 Out-of-Plane Wall Anchorage to Flexible Diaphragms

This section shall apply in Seismic Zone 4 where flexible diaphragms, as defined in Section 208.5.1.3, provide lateral support for walls.

1. Elements of the wall anchorage system shall be designed for the forces specified in Section 208.9 where $R_p = 3.0$ and $a_p = 1.5$.
2. In Seismic Zone 4, the value of F_p used for the design of the elements of the wall anchorage system shall not be less than 6.1 kN per lineal meter of wall substituted for E .
3. See Section 206.4 for minimum design forces in other seismic zones.

4. When elements of the wall anchorage system are not loaded concentrically or are not perpendicular to the wall, the system shall be designed to resist all components of the forces induced by the eccentricity.
5. When pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall be that specified in Section 208.7.2.7, Item 2.
6. The strength design forces for steel elements of the wall anchorage system shall be 1.4 times the forces otherwise required by this section.
7. The strength design forces for wood elements of the wall anchorage system shall be 0.85 times the force otherwise required by this section and these wood elements shall have a minimum actual net thickness of 63.5 mm.

208.7.2.7 Diaphragms

1. The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.
2. Floor and roof diaphragms shall be designed to resist the forces determined in accordance with the following equation:

$$F_{px} = \frac{F_t + \sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px} \quad (208-22)$$

The force F_{px} determined from Equation 208-22 need not exceed $1.0C_a I w_{px}$, but shall not be less than $0.5C_a I w_{px}$.

When the diaphragm is required to transfer design seismic forces from the vertical-resisting elements above the diaphragm to other vertical-resisting elements below the diaphragm due to offset in the placement of the elements or to changes in stiffness in the vertical elements, these forces shall be added to those determined from Equation 208-22.

3. Design seismic forces for flexible diaphragms providing lateral supports for walls or frames of masonry or concrete shall be determined using Equation 208-22 based on the load determined in

accordance with Section 208.5.2 using a R not exceeding 4.

4. Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 208.7.2.7. Added chords of subdiaphragms may be used to form subdiaphragms to transmit the anchorage forces to the main continuous crossties. The maximum length-to-width ratio of the wood structural sub-diaphragm shall be $2\frac{1}{2}:1$.
5. Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Section 208.7.2.7. In Seismic Zone 2 and 4, anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, wood ledgers or framing shall not be used in cross-grain bending or cross-grain tension, and the continuous ties required by Item 4 shall be in addition to the diaphragm sheathing.
6. Connections of diaphragms to the vertical elements in structures in Seismic Zone 4, having a plan irregularity of Type 1, 2, 3 or 4 in Table 208-10, shall be designed without considering either the one-third increase or the duration of load increase considered in allowable stresses for elements resisting earthquake forces.
7. In structures in Seismic Zone 4 having a plan irregularity of Type 2 in Table 208-10, diaphragm chords and drag members shall be designed considering independent movement of the projecting wings of the structure. Each of these diaphragm elements shall be designed for the more severe of the following two assumptions:
 - a. Motion of the projecting wings in the same direction.
 - b. Motion of the projecting wings in opposing directions.

Exception:

This requirement may be deemed satisfied if the procedures of Section 208.5.3 in conjunction with a three-dimensional model have been used to determine the lateral seismic forces for design.

208.7.2.8 Framing below the Base

The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure. The special detailing requirements of Chapters 4, 5 and 7, as appropriate, shall apply to columns

supporting discontinuous lateral-force-resisting elements and to SMRF, IMRF, EBF, STMF and MMRWF system elements below the base, which are required to transmit the forces resulting from lateral loads to the foundation.

208.7.2.9 Building Separations

All structures shall be separated from adjoining structures. Separations shall allow for the displacement Δ_m . Adjacent buildings on the same property shall be separated by at least Δ_{MT} where

$$\Delta_{MT} = \sqrt{(\Delta_{M1}) + (\Delta_{M2})^2} \quad (208-23)$$

and Δ_{M1} and Δ_{M2} are the displacements of the adjacent buildings.

When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least the displacement Δ_M of that structure.

Exception:

Smaller separations or property line setbacks may be permitted when justified by rational analyses based on maximum expected ground motions.

208.8 Non-Building Structures

208.8.1 General

208.8.1.1 Scope

Non-building structures include all self-supporting structures other than buildings that carry gravity loads and resist the effects of earthquakes. Non-building structures shall be designed to provide the strength required to resist the displacements induced by the minimum lateral forces specified in this section. Design shall conform to the applicable provisions of other sections as modified by the provisions contained in Section 208.8.

208.8.1.2 Criteria

The minimum design seismic forces prescribed in this section are at a level that produces displacements in a fixed base, elastic model of the structure, comparable to those expected of the real structure when responding to the Design Basis Ground Motion. Reductions in these forces using the coefficient R is permitted where the design of non-building structures provides sufficient strength and ductility, consistent with the provisions specified herein for buildings, to resist the effects of seismic ground motions as represented by these design forces.

When applicable, design strengths and other detailed design criteria shall be obtained from other sections or their referenced standards. The design of non-building structures shall use the load combinations or factors specified in Section 203.3 or 203.4. For non-building structures designed using Section 208.8.3, 208.8.4 or 208.8.5, the Reliability/Redundancy Factor, ρ , may be taken as 1.0.

When applicable design strengths and other design criteria are not contained in or referenced by this code, such criteria shall be obtained from approved national standards.

208.8.1.3 Weight W

The weight, W , for non-building structures shall include all dead loads as defined for buildings in Section 208.6.1. For purposes of calculating design seismic forces in non-building structures, W shall also include all normal operating contents for items such as tanks, vessels, bins and piping.

208.8.1.4 Period

The fundamental period of the structure shall be determined by rational methods such as by using Method B in Section 208.5.2.2.

208.8.1.5 Drift

The drift limitations of Section 208.6.5 need not apply to non-building structures. Drift limitations shall be established for structural or nonstructural elements whose failure would cause life hazards. $P\Delta$ effects shall be considered for structures whose calculated drifts exceed the values in Section 208.6.3.

Table 208-12 R and Ω_o Factors for Non-building Structures

STRUCTURE TYPE	R	Ω_o
1. Vessels, including tanks and pressurized spheres, on braced or unbraced legs.	2.2	2.0
2. Cast-in-place concrete silos and chimneys having walls continuous to the foundations	3.6	2.0
3. Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels.	2.9	2.0
4. Trussed towers (freestanding or guyed), guyed stacks and chimneys.	2.9	2.0
5. Cantilevered column-type structures.	2.2	2.0
6. Cooling towers.	3.6	2.0
7. Bins and hoppers on braced or unbraced legs.	2.9	2.0
8. Storage racks.	3.6	2.0
9. Signs and billboards.	3.6	2.0
10. Amusement structures and monuments.	2.2	2.0
11. All other self-supporting structures not otherwise covered.	2.9	2.0

208.8.1.6 Interaction Effects

In Seismic Zone 4, structures that support flexible nonstructural elements whose combined weight exceeds 25 percent of the weight of the structure shall be designed considering interaction effects between the structure and the supported elements.

208.8.2 Lateral Force

Lateral-force procedures for non-building structures with structural systems similar to buildings (those with structural systems which are listed in Table 208-11) shall be selected in accordance with the provisions of Section 208.4.

Exception:

Intermediate moment-resisting frames (IMRF) may be used in Seismic Zone 4 for non-building structures in Occupancy Categories III and IV if (1) the structure is less than 15 m in height and (2) the value R used in reducing

calculated member forces and moments does not exceed 2.8.

208.8.3 Rigid Structures

Rigid structures (those with period T less than 0.06 s) and their anchorages shall be designed for the lateral force obtained from Equation 208-24.

$$V = 0.7C_aIW \quad (208-24)$$

The force V shall be distributed according to the distribution of mass and shall be assumed to act in any horizontal direction.

208.8.4 Tanks with Supported Bottoms

Flat bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist the seismic forces calculated using the procedures in Section 208.9 for rigid structures considering the entire weight of the tank and its contents. Alternatively, such tanks may be designed using one of the two procedures described below:

1. A response spectrum analysis that includes consideration of the actual ground motion anticipated at the site and the inertial effects of the contained fluid.
2. A design basis prescribed for the particular type of tank by an approved national standard, provided that the seismic zones and occupancy categories shall be in conformance with the provisions of Sections 208.4.4.2 and 208.4.4.3, respectively.

208.8.5 Other Non-building Structures

Non-building structures that are not covered by Section 208.8.3 and 208.8.4 shall be designed to resist design seismic forces not less than those determined in accordance with the provisions in Section 208.5 with the following additions and exceptions:

1. The factors R and Ω_o shall be as set forth in Table 208-12. The total design base shear determined in accordance with Section 208.5.2 shall not be less than the following:

$$V = 0.56C_aIW \quad (208-25)$$

Additionally, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{1.6ZN_vI}{R}W \quad (208-26)$$

- The vertical distribution of the design seismic forces in structures covered by this section may be determined by using the provisions of Section 208.5.2.3 or by using the procedures of Section 208.5.3.

Exception:

For irregular structures assigned to Occupancy Categories I and II that cannot be modeled as a single mass, the procedures of Section 208.5.3 shall be used.

- Where an approved national standard provides a basis for the earthquake-resistant design of a particular type of non-building structure covered by this section, such a standard may be used, subject to the limitations in this section:

The seismic zones and occupancy categories shall be in conformance with the provisions of Sections 208.4.4 and 208.4.2, respectively.

The values for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the values that would be obtained using these provisions.

208.9 Lateral Force on Elements of Structures, Nonstructural Components and Equipment Supported by Structures

208.9.1 General

Elements of structures and their attachments, permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure shall be designed to resist the total design seismic forces prescribed in Section 208.9.2.

Attachments for floor- or roof-mounted equipment weighing less than 1.8 kN, and furniture need not be designed.

Attachments shall include anchorages and required bracing. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

When the structural failure of the lateral-force-resisting systems of non-rigid equipment would cause a life hazard, such systems shall be designed to resist the seismic forces prescribed in Section 208.9.2.

When permissible design strengths and other acceptance criteria are not contained in or referenced by this code, such criteria shall be obtained from approved national standards subject to the approval of the building official.

208.9.2 Design for Total Lateral Force

The total design lateral seismic force, F_p , shall be determined from the following equation:

$$F_p = 4C_a I_p W_p \quad (208-27)$$

Alternatively, F_p may be calculated using the following equation:

$$F_p = \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_y} \right) W_p \quad (208-28)$$

Except that F_p shall not be less than $0.7C_a I_p W_p$ and need not be more than $4C_a I_p W_p$.

where

- h_x = the element or component attachment elevation with respect to grade.
 h_x shall not be taken less than 0.0.
- h_r = the structure roof elevation with respect to grade.
- a_p = the in-structure Component Amplification Factor that varies from 1.0 to 2.5.

A value for a_p shall be selected from Table 208-13. Alternatively, this factor may be determined based on the dynamic properties or empirical data of the component and the structure that supports it. The value shall not be taken less than 1.0.

R_p is the Component Response Modification Factor that shall be taken from Table 208-13, except that R_p for anchorages shall equal 1.5 for shallow expansion anchor bolts, shallow chemical anchors or shallow cast-in-place anchors. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. When anchorage is constructed of non-ductile materials, or by use of adhesive, R_p shall equal 1.0.

The design lateral forces determined using Equation 208-27 or 208-19 shall be distributed in proportion to the mass distribution of the element or component.

Forces determined using Equation 208-27 or 208-28 shall be used to design members and connections that transfer those forces to the seismic-resisting systems. Members and connection design shall use the load combinations and factors specified in Section 203.3 or 203.4. The Reliability/Redundancy Factor, ρ , may be taken equal to 1.0.

For applicable forces and Component Response Modification Factors in connectors for exterior panels and diaphragms, refer to Sections 208.7.2.3 and 208.7.2.7.

Forces shall be applied in the horizontal directions, which result in the most critical loadings for design.

Table 208-13 Horizontal Force Factors, a_p and R_p for Elements of Structures and Nonstructural Components and Equipment

Category	Element or Component	a_p	R_p	Footnote
1. Elements of Structures	1. Walls including the following			
	a. Unbraced (cantilevered) parapets	2.5	3.0	
	b. Exterior walls at or above the ground floor and parapets braced above their centers of gravity	1.0	3.0	2
	c. All interior-bearing and non-bearing walls	1.0	3.0	2
	2. Penthouse (except when framed by an extension of the structural frame)	2.5	4.0	
	3. Connections for prefabricated structural elements other walls. See also Section 208.7.2	1.0	3.0	3

Table 208-13 (continued)

Category	Element or Component	a_p	R_p	Footnote
2. Nonstructural Components	1. Exterior and interior ornamentations and appendages.			
	a. Laterally braced or anchored to the structural frame at a point below their centers of mass	2.5	3.0	
	b. Laterally braced or anchored to the structural frame at or above their centers of mass	1.0	3.0	
	2. Signs and billboards	2.5	3.0	
	3. Storage racks (include contents) over 1.8 m tall.	2.5	4.0	4
	4. Permanent floor-supported cabinets and book stacks more than 1.8 m in height (include contents)	1.0	3.0	5
	5. Anchorage and lateral bracing for suspended ceilings and light fixtures	1.0	3.0	3, 6, 7, 8
	6. Access floor systems	1.0	3.0	4, 5, 9
	7. Masonry or concrete fences over 1.8 m high	1.0	3.0	
	8. Partitions.	1.0	3.0	

Table 208-13 (continued)

Category	Element or Component	a_p	R_p	Footnote
3. Equipment	1. Tanks and vessels (include contents), including support systems.	1.0	3.0	
	2. Electrical, mechanical and plumbing equipment and associated conduit and ductwork and piping.	1.0	3.0	5, 10, 11, 12, 13, 14, 15, 16
	3. Any flexible equipment laterally braced or anchored to the structural frame at a point below their center of mass	2.5	3.0	5, 10, 14, 15, 16
	4. Anchorage of emergency power supply systems and essential communications equipment. Anchorage and support systems for battery racks and fuel tanks necessary for operation of emergency equipment. See also Section 208.7.2	1.0	3.0	17, 18
	5. Temporary containers with flammable or hazardous materials.	1.0	3.0	19

Table 208-13 (continued)

Category	Element or Component	a_p	R_p	Footnote
4. Other Components	1. Rigid components with ductile material and attachments.	1.0	3.0	1
	2. Rigid components with nonductile material or attachments	1.0	1.5	1
	3. Flexible components with ductile material and attachments.	2.5	3.0	1
	4. Flexible components with nonductile material or attachments.	2.5	1.5	1

Notes for Table 208.13

- ¹ See Section 208.2 for definitions of flexible components and rigid components.
- ² See Section 208.8.7.2.3 and 208.7.2.7 for concrete and masonry walls and Section 208.9.2 for connections for panel connectors for panels.
- ³ Applies to Seismic Zones 2 and 4 only.
- ⁴ Ground supported steel storage racks may be designed using the provisions of Sections 208.8. Load and resistance factor design may be used for the design of cold-formed steel members, provided seismic design forces are equal to or greater than those specified in Section 208.9.2 or 208.8.3 as appropriate.
- ⁵ Only anchorage or restraints need be designed.
- ⁶ Ceiling weight shall include all light fixtures and other equipment or partitions that are laterally supported by the ceiling. For purposes of determining the seismic force, a ceiling weight of not less than 0.2 kPa shall be used.
- ⁷ Ceilings constructed of lath and plaster or gypsum board screw or nail attached to suspended members that support a ceiling at one level extending from wall to wall need not be analyzed, provided the walls are not over 15 meters apart.
- ⁸ Light fixtures and mechanical services installed in metal suspension systems for acoustical tile and lay-in panel ceilings shall be independently supported from

the structure above as specified in UBC Standard 25-2, Part III.

- ⁹ W_p for access floor systems shall be the dead load of the access floor system plus 25 percent of the floor live load plus a 0.5 kPa partition load allowance.
- ¹⁰ Equipment includes, but is not limited to, boilers, chillers, heat exchangers, pumps, air-handling units, cooling towers, control panels, motors, switchgear, transformers and life-safety equipment. It shall include major conduit, ducting and piping, which services such machinery and equipment and fire sprinkler systems. See Section 208.9.2 for additional requirements for determining a_p for nonrigid or flexibly mounted equipment.
- ¹¹ Seismic restraints may be omitted from piping and duct supports if all the following conditions are satisfied:
 - ^{11.1} Lateral motion of the piping or duct will not cause damaging impact with other systems.
 - ^{11.2} The piping or duct is made of ductile material with ductile connections.
 - ^{11.3} Lateral motion of the piping or duct does not cause impact of fragile appurtenances (e.g., sprinkler heads) with any other equipment, piping or structural member.
 - ^{11.4} Lateral motion of the piping or duct does not cause loss of system vertical support.
 - ^{11.5} Rod-hung supports of less than 300 mm in length have top connections that cannot develop moments.
 - ^{11.6} Support members cantilevered up from the floor are checked for stability.
- ¹² Seismic restraints may be omitted from electrical raceways, such as cable trays, conduit and bus ducts, if all the following conditions are satisfied:
 - ^{12.1} Lateral motion of the raceway will not cause damaging impact with other systems.
 - ^{12.2} Lateral motion of the raceway does not cause loss of system vertical support.
 - ^{12.3} Rod-hung supports of less than 300 mm in length have top connections that cannot develop moments.
 - ^{12.4} Support members cantilevered up from the floor are checked for stability.
- ¹³ Piping, ducts and electrical raceways, which must be functional following an earthquake, spanning between different buildings or structural systems shall be sufficiently flexible to withstand relative motion of support points assuming out-of-phase motions.
- ¹⁴ Vibration isolators supporting equipment shall be designed for lateral loads or restrained from displacing laterally by other means. Restraint shall also be provided, which limits vertical displacement, such that lateral restraints do not become disengaged. a_p and R_p for equipment supported on vibration isolators shall be taken as 2.5 and 1.5, respectively, except that if the isolation mounting frame is supported by shallow or

expansion anchors, the design forces for the anchors calculated by Equations 208-27, or 208-28 (including limits), shall be additionally multiplied by factor of 2.0.

- ¹⁵ Equipment anchorage shall not be designed such that loads are resisted by gravity friction (e.g., friction clips).
- ¹⁶ Expansion anchors, which are required to resist seismic loads in tension, shall not be used where operational vibrating loads are present.
- ¹⁷ Movement of components within electrical cabinets, rack-and-skid-mounted equipment and portions of skid-mounted electromechanical equipment that may cause damage to other components by displacing, shall be restricted by attachment to anchored equipment or support frames.
- ¹⁸ Batteries on racks shall be restrained against movement in all direction due to earthquake forces.
- ¹⁹ Seismic restraints may include straps, chains, bolts, barriers or other mechanisms that prevent sliding, falling and breach of containment of flammable and toxic materials. Friction forces may not be used to resist lateral loads in the restraints unless positive uplift restraint is provided which ensures that the friction forces act continuously.

208.9.3 Specifying Lateral Forces

Design specifications for equipment shall either specify the design lateral forces prescribed herein or reference these provisions.

208.9.4 Relative Motion of Equipment Attachments

For equipment in Categories I and II buildings as defined in Table 103-1, the lateral-force design shall consider the effects of relative motion of the points of attachment to the structure, using the drift based upon Δ_M .

208.9.5 Alternative Designs

Where an approved national standard or approved physical test data provide a basis for the earthquake-resistant design of a particular type of equipment or other nonstructural component, such a standard or data may be accepted as a basis for design of the items with the following limitations:

1. These provisions shall provide minimum values for the design of the anchorage and the members and connections that transfer the forces to the seismic-resisting system.
2. The force, F_p , and the overturning moment used in the design of the nonstructural component shall not be less than 80 percent of the values that would be obtained using these provisions.

208.10 Alternative Earthquake Load Procedure

The earthquake load procedure of latest edition of ASCE/SEI 7 prior to the release of this code may be used in determining the earthquake loads as an alternative procedure subject to reliable research work commissioned by the owner or the engineer-on-record to provide for all data required due to the non-availability of PHIVOLCS-issued spectral acceleration maps for all areas in the Philippines.

The engineer-on-record shall be responsible for the spectral acceleration and other related data not issued by PHIVOLCS used in the determination of the earthquake loads. This alternative earthquake load procedure shall be subject to Peer Review and approval of the Building Official.

SECTION 209
SOIL LATERAL LOADS
209.1 General

Basement, foundation and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 209.1 shall be used as the minimum design lateral soil loads unless specified otherwise in a soil investigation report approved by the building official. Basement walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top are permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils with expansion potential are present at the site.

Exception:

Basement walls extending not more than 2.4 m below grade and supporting flexible floor systems shall be permitted to be designed for active pressure.

Table 209-1 - Soil Lateral Load

Description of Backfill Material ^c	Unified Soil Classification	Design Lateral Soil Load ^a kPa per m depth	
		Active pressure	At-rest pressure
Well-graded, clean gravels; gravel-sand mixes	GW	5	10
Poorly graded clean gravels; gravel-sand mixes	GP	5	10
Silty gravels, poorly graded gravel-sand mixes	GM	6	10
Clayey gravels, poorly graded gravel-and-clay mixes	GC	7	10
Well-graded, clean sands; gravelly sand mixes	SW	5	10
Poorly graded clean sands; sand-gravel mixes	SP	5	10
Silty sands, poorly graded sand-silt mixes	SM	7	10
Sand-silt clay mix with plastic fines	SM-SC	7	16
Clayey sands, poorly graded sand-clay mixes	SC	10	16
Inorganic silts and clayey silts	ML	7	16
Mixture of inorganic silt and clay	ML-CL	10	16
Inorganic clays of low to medium plasticity	CL	10	16
Organic silts and silt clays, low plasticity	OL	Note b	Note b
Inorganic clayey silts, elastic silts	MH	Note b	Note b
Inorganic clays of high plasticity	CH	Note b	Note b
Organic clays and silty clays	OH	Note b	Note b

^a Design lateral soil loads are given for moist conditions for the specified soils at their optimum densities. Actual field conditions shall

govern. Submerged or saturated soil pressures shall include the weight of the buoyant soil plus the hydrostatic loads.

^b Unsuitable as backfill material.

^c The definition and classification of soil materials shall be in accordance with ASTM D 2487.

SECTION 210 RAIN LOADS

210.1 Roof Drainage

Roof drainage systems shall be designed in accordance with the provisions of the code having jurisdiction in the area. The flow capacity of secondary (overflow) drains or scuppers shall not be less than that of the primary drains or scuppers.

210.2 Design Rain Loads

Each portion of a roof shall be designed to sustain the load of rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow.

$$R = 0.009(8d_s + d_h) \quad (210-1)$$

where

d_h = additional depth of water on the undeflected roof above the inlet of secondary drainage system at its design flow (i.e., the hydraulic head), mm

d_s = depth of water on the undeflected roof up to the inlet of secondary drainage system when the primary drainage system is blocked (i.e., the static head), mm

R = rain load on the undeflected roof, kPa

When the phrase “undeflected roof” is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.

210.3 Ponding Instability

For roofs with a slope less than 6 mm per 300 mm, the design calculations shall include verification of adequate stiffness to preclude progressive deflection.

210.4 Controlled Drainage

Roofs equipped with hardware to control the rate of drainage shall be equipped with a secondary drainage system at a higher elevation that limits accumulation of water on the roof above that elevation. Such roofs shall be designed to sustain the load of rainwater that will accumulate on them to the elevation of the secondary drainage system plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow determined from Section 210.2. Such roofs shall also be checked for ponding instability in accordance with Section 210.3.

SECTION 211 FLOOD LOADS

211.1 General

Within flood hazard areas as established in Section 211.3, all new construction of buildings, structures and portions of buildings and structures, including substantial improvement and restoration of substantial damage to buildings and structures, shall be designed and constructed to resist the effects of flood hazards and flood loads. For buildings that are located in more than one flood hazard area, the provisions associated with the most restrictive flood hazard area shall apply.

211.2 Definitions

The following words and terms shall, for the purposes of this section, have the meanings shown herein.

BASE FLOOD refers to flood having a 1-percent chance of being equaled or exceeded in any given year.

BASE FLOOD ELEVATION (BFE) is the elevation of the base flood, m, including wave height, relative to the datum to be set by the specific national or local government agency.

BASEMENT is the portion of a building having its floor subgrade (below ground level) on all sides.

DESIGN FLOOD is the flood associated with the greater of the following two areas:

1. Area with a flood plain subject to a 1-percent or greater chance of flooding in any year; or
2. Area designated as a flood hazard area on a community's flood hazard map, or otherwise legally designated.

DESIGN FLOOD ELEVATION (DFE) is the elevation of the “design flood,” including wave height, m, relative to the datum specified on the community's legally designated flood hazard map. The design flood elevation shall be the elevation of the highest existing grade of the perimeter of the building plus the depth specified on the flood hazard map.

DRY FLOODPROOFING is a combination of design modifications that results in a building or structure, including the attendant utility and sanitary facilities, being water tight with walls substantially impermeable to the passage of water and with structural components having the capacity to resist loads as identified in the code.

EXISTING CONSTRUCTION refers to buildings and structures for which the “start of construction” commenced before the effective date of the ordinance or standard. “Existing construction” is also referred to as “existing structures.”

EXISTING STRUCTURE See “Existing construction.”

FLOOD or FLOODING is a general and temporary condition of partial or complete inundation of normally dry land from:

1. The overflow of inland or tidal waters.
2. The unusual and rapid accumulation or runoff of surface waters from any source.

FLOOD DAMAGE-RESISTANT MATERIALS are construction material capable of withstanding direct and prolonged contact with floodwaters without sustaining any damage that requires more than cosmetic repair.

FLOOD HAZARD AREA refers to the greater of the following two areas:

1. The area within a flood plain subject to a 1-percent or greater chance of flooding in any year.
2. The area designated as a flood hazard area on a community’s flood hazard map, or otherwise legally designated.

FLOOD HAZARD AREA SUBJECT TO HIGH VELOCITY-WAVE ACTION refers to area within the flood hazard area that is subject to high velocity wave action.

FLOODWAY is the channel of the river, creek or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height.

LOWEST FLOOR refers to the floor of the lowest enclosed area, including basement, but excluding any unfinished or flood-resistant enclosure, usable solely for vehicle parking, building access or limited storage provided that such enclosure is not built so as to render the structure in violation of this section.

START OF CONSTRUCTION refers to the date of permit issuance for new construction and substantial improvements to existing structures, provided the actual start of construction, repair, reconstruction, rehabilitation, addition, placement or other improvement is within 180 days after the date of issuance. The actual start of

construction means the first placement of permanent construction of a building (including a manufactured home) on a site, such as the pouring of a slab or footings, installation of pilings or construction of columns. Permanent construction does not include land preparation (such as clearing, excavation, grading or filling), the installation of streets or walkways, excavation for a basement, footings, piers or foundations, the erection of temporary forms or the installation of accessory buildings such as garages or sheds not occupied as dwelling units or not part of the main building. For a substantial improvement, the actual “start of construction” means the first alteration of any wall, ceiling, floor or other structural part of a building, whether or not that alteration affects the external dimensions of the building.

SUBSTANTIAL DAMAGE refers to damage to any origin sustained by a structure whereby the cost of restoring the structure to its before-damaged condition would equal or exceed 50 percent of the market value of the structure before the damage occurred.

SUBSTANTIAL IMPROVEMENT refers to any repair, reconstruction, rehabilitation, addition or improvement of a building or structure, the cost of which equals or exceeds 50 percent of the market value of the structure before the improvement or repair is started. If the structure has sustained substantial damage, any repairs are considered substantial improvement regardless of the actual repair work performed. The term does not, however, include either:

1. Any project for improvement of a building required to correct existing health, sanitary or safety code violations identified by the building official and that are the minimum necessary to assure safe living conditions.
2. Any alteration of a historic structure provided that the alteration will not preclude the structure’s continued designation as a historic structure.

211.3 Design Requirements

211.3.1 Design Loads

Structural systems of buildings or other structures shall be designed, constructed, connected, and anchored to resist flotation, collapse, and permanent lateral displacement due to action of flood loads associated with the design flood (see Section 211.3.3) and other loads in accordance with the load combinations of Section 203.

211.3.2 Erosion and Scour

The effects of erosion and scour shall be included in the calculation of loads on buildings and other structures in flood hazard areas.

211.3.3 Loads on Breakaway Walls

Walls and partitions required by ASCE/SEI 24, to break away, including their connections to the structure, shall be designed for the largest of the following loads acting perpendicular to the plane of the wall:

1. The wind load specified in Section 207.
2. The earthquake load specified in Section 208.
3. 0.48 kPa.

The loading at which breakaway walls are intended to collapse shall not exceed 0.96 kPa unless the design meets the following conditions:

1. Breakaway wall collapse is designed to result from a flood load less than that which occurs during the base flood.
2. The supporting foundation and the elevated portion of the building shall be designed against collapse, permanent lateral displacement, and other structural damage due to the effects of flood loads in combination with other loads as specified in Section 203.

211.4 Loads During Flooding

211.4.1 Load Basis

In flood hazard areas, the structural design shall be based on the design flood.

211.4.2 Hydrostatic Loads

Hydrostatic loads caused by a depth of water to the level of the DFE shall be applied over all surfaces involved, both above and below ground level, except that for surfaces exposed to free water, the design depth shall be increased by 0.30 m. Reduced uplift and lateral loads on surfaces of enclosed spaces below the DFE shall apply only if provision is made for entry and exit of floodwater.

211.4.3 Hydrodynamic Loads

Dynamic effects of moving water shall be determined by a detailed analysis utilizing basic concepts of fluid mechanics.

Exception:

Where water velocities do not exceed 3.05 m/s, dynamic effects of moving water shall be permitted to be converted into equivalent hydrostatic loads by increasing the DFE for design purposes by an equivalent surcharge depth, d_h , on the headwater side and above the ground level only, equal to

$$d_h = \frac{aV^2}{2g} \quad (211-1)$$

where

- V = average velocity of water, m/s
 g = acceleration due to gravity, 9.81 m/s²
 a = coefficient of drag or shape factor (not less than 1.25)

The equivalent surcharge depth shall be added to the DFE design depth and the resultant hydrostatic pressures applied to, and uniformly distributed across, the vertical projected area of the building or structure that is perpendicular to the flow. Surfaces parallel to the flow or surfaces wetted by the tail water shall be subject to the hydrostatic pressures for depths to the DFE only.

211.4.4 Wave Loads

Wave loads shall be determined by one of the following three methods: (1) by using the analytical procedures outlined in this section, (2) by more advanced numerical modeling procedures, or (3) by laboratory test procedures (physical modeling).

Wave loads are those loads that result from water waves propagating over the water surface and striking a building or other structure. Design and construction of buildings and other structures subject to wave loads shall account for the following loads: waves breaking on any portion of the building or structure; uplift forces caused by shoaling waves beneath a building or structure, or portion thereof; wave run-up striking any portion of the building or structure; wave-induced drag and inertia forces; and wave-induced scour at the base of a building or structure, or its foundation. Wave loads shall be included for both V-Zones and A-Zones. In V-Zones, waves are 0.91 m high, or higher; in coastal floodplains landward of the V-Zone, waves are less than 0.91 m.

Nonbreaking and broken wave loads shall be calculated using the procedures described in Sections 211.4.2 and 211.4.3 that show how to calculate hydrostatic and hydrodynamic loads.

Breaking wave loads shall be calculated using the procedures described in Sections 211.4.4.1 through 211.4.4.4. Breaking wave heights used in the procedures described in Sections 211.4.4.1 through 211.4.4.4 shall be calculated for V-Zones and Coastal A-Zones using Equations 211-2 and 211-3.

$$H_b = 0.78d_s \quad (211-2)$$

where

H_b = breaking wave height, m
 d_s = local still water depth, m

The local still water depth shall be calculated using Equation 211-3, unless more advanced procedures or laboratory tests permitted by this section are used.

$$d_s = 0.65(BFE - G) \quad (211-3)$$

where

G = ground elevation, m

211.4.4.1 Breaking Wave Loads on Vertical Pilings and Columns

The net force resulting from a breaking wave acting on a rigid vertical pile or column shall be assumed to act at the still water elevation and shall be calculated by the following:

$$F_D = 0.5\gamma_\omega C_D D H_b^2 \quad (211-4)$$

where

F_D = net wave force, kN
 γ_ω = unit weight of water, in lb per cubic kN/m³, = 9.80 kN/m³ for fresh water and 10.05 kN/m³ for salt water
 C_D = coefficient of drag for breaking waves, = 1.75 for round piles or columns, and = 2.25 for square piles or columns
 D = pile or column diameter, m for circular sections, or for a square pile or column, 1.4 times the width of the pile or column, m
 H_b = breaking wave height, m

211.4.4.2 Breaking Wave Loads on Vertical Walls

Maximum pressures and net forces resulting from a normally incident breaking wave (depth-limited in size, with $H_b = 0.78d_s$) acting on a rigid vertical wall shall be calculated by the following:

$$P_{max} = C_p \gamma_\omega d_s + 1.2 \gamma_\omega d_s \quad (211-5)$$

and

$$F_t = 1.1 C_p \gamma_\omega d_s^2 + 2.4 \gamma_\omega d_s^2 \quad (211-6)$$

where

P_{max} = maximum combined dynamic ($C_p \gamma_\omega d_s$) and static ($1.2 \gamma_\omega d_s$) wave pressures, also referred to as shock pressures, kN/m²
 F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse, or wave impact force, kN/m, acting near the still water elevation
 C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 211-1)
 γ_ω = unit weight of water, kN/m³, 9.80 kN/m³ for fresh water and 10.05 kN/m³ for salt water
 d_s = still water depth, m at base of building or other structure where the wave breaks

This procedure assumes the vertical wall causes a reflected or standing wave against the waterward side of the wall with the crest of the wave at a height of 1.2d, above the still water level. Thus, the dynamic static and total pressure distributions against the wall are as shown in Figure 211-1.

This procedure also assumes the space behind the vertical wall is dry, with no fluid balancing the static component of the wave force on the outside of the wall. If free water exists behind the wall, a portion of the hydrostatic component of the wave pressure and force disappears (see Figure 211-2) and the net force shall be computed by Equation 211-7 (the maximum combined wave pressure is still computed with Equation 211-5).

$$F_t = 1.1 C_p \gamma_\omega d_s^2 + 1.9 \gamma_\omega d_s^2 \quad (211-7)$$

where

F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse, or wave impact force, kN/m, acting near the still water elevation

C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 211-1)

γ_w = unit weight of water, kN/m^3 , = 9.80 kN/m^3 for fresh water and 10.05 kN/m^3 for salt water

d_s = still water depth, m at base of building or other structure where the wave breaks

211.4.4.3 Breaking Wave Loads on Non-vertical Walls

Breaking wave forces given by Equations 211-6 and 211-7 shall be modified in instances where the walls or surfaces upon which the breaking waves act are non-vertical. The horizontal component of breaking wave force shall be given by

$$F_{nv} = F_t \sin^2 \alpha \quad (211-8)$$

where

F_{nv} = horizontal component of breaking wave force, kN/m

F_t = net breaking wave force acting on a vertical surface, kN/m

α = vertical angle between non-vertical surface and the horizontal

211.4.4.4 Breaking Wave Loads from Obliquely Incident Waves

Breaking wave forces given by Equations 211-6 and 211-7 shall be modified in instances where waves are obliquely incident. Breaking wave forces from non-normally incident waves shall be given by

$$F_{oi} = F_t \sin^2 \alpha \quad (211-9)$$

where

F_{oi} = horizontal component of obliquely incident breaking wave force, kN/m

α = net breaking wave force (normally incident waves) acting on a vertical surface, kN/m

211.4.5 Impact Loads

Impact loads are those that result from debris, ice, and any object transported by floodwaters striking against buildings and structures, or parts thereof. Impact loads shall be determined using a rational approach as concentrated loads acting horizontally at the most critical location at or below the DFE.

211.5 Establishment of Flood Hazard Areas

To establish flood hazard areas, the governing body shall adopt a flood hazard map and supporting data. The flood hazard map shall include, at a minimum, areas of special flood hazard where records are available.

211.6 Design and Construction

The design and construction of buildings and structures located in flood hazard areas, including flood hazard areas subject to high velocity wave action.

211.7 Flood Hazard Documentation

The following documentation shall be prepared and sealed by an engineer-of-record and submitted to the building official:

1. For construction in flood hazard areas not subject to high-velocity wave action:
 - 1.1 The elevation of the lowest floor, including the basement, as required by the lowest floor elevation.
 - 1.2 For fully enclosed areas below the design flood elevation where provisions to allow for the automatic entry and exit of floodwaters do not meet the minimum requirements, construction documents shall include a statement that the design will provide for equalization of hydrostatic flood forces.
 - 1.3 For dry flood-proofed nonresidential buildings, construction documents shall include a statement that the dry flood-proofing is designed.
2. For construction in flood hazard areas subject to high-velocity wave action:
 - 2.1 The elevation of the bottom of the lowest horizontal structural member as required by the lowest floor elevation.
 - 2.2 Construction documents shall include a statement that the building is designed, including that the pile or column foundation and building or structure to be attached thereto is designed to be anchored to resist flotation, collapse and lateral movement due to the effects of wind and flood loads acting simultaneously on all building components, and other load requirements of Section 203.

- 2.3 For breakaway walls designed to resist a nominal load of less than 0.5 kPa or more than 1.0 kPa, construction documents shall include a statement that the breakaway wall is designed.

Table 211-1 Value of Dynamic Pressure Coefficient, C_p

Building Category	C_p
I	1.6
II	2.8
II	3.2
IV	3.5

ASCE/SEI

American Society of Civil Engineers
Structural Engineering Institute
1801 Alexander Bell Drive
Reston, VA 20191-4400

ASCE/SEI 24

Section 5.3.3

Flood Resistant Design and Construction, 1998

211.8 Consensus Standards and Other Referenced Documents

This section lists the consensus standards and other documents which are adopted by reference within this chapter:

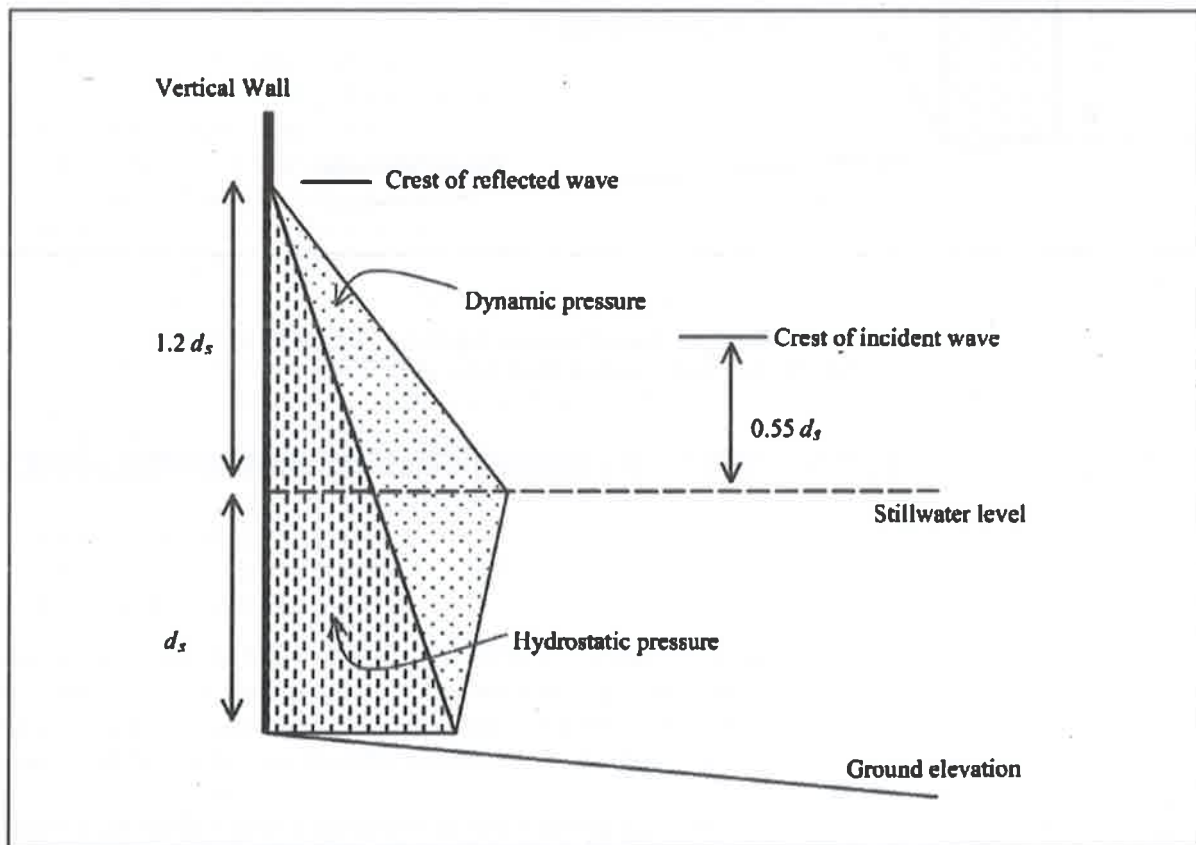


Figure 211-1
Normally Incident Breaking Wave Pressures Against a Vertical Wall
(Space Behind Vertical Wall Is Dry)

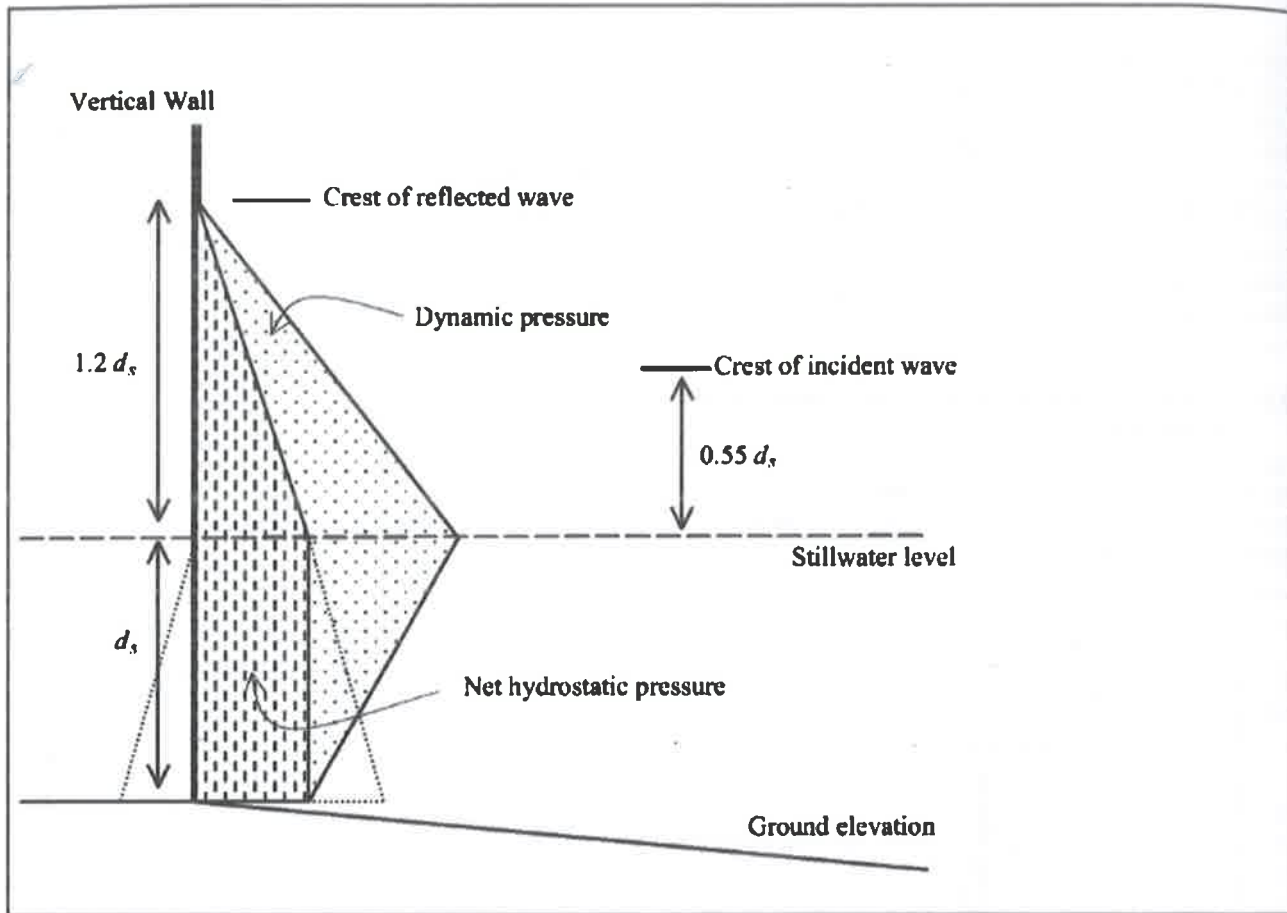


Figure 211-2
Normally Incident Breaking Wave Pressures Against a Vertical Wall
(Still Water Level Equal on Both Sides of Wall)

Chapter 3

EARTHWORKS AND FOUNDATIONS

**NATIONAL STRUCTURAL CODE OF THE PHILIPPINES
VOLUME I**

**BUILDINGS, TOWERS AND
OTHER VERTICAL STRUCTURES**

SEVENTH EDITION, 2015

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Table of Contents

SECTION 301 – GENERAL REQUIREMENTS	3-3
301.1 Scope.....	3-3
301.2 Quality and Design.....	3-3
301.3 Allowable Bearing Pressures.....	3-3
301.4 Definitions.....	3-3
SECTION 302 - EXCAVATION AND FILLS.....	3-3
302.1 General	3-3
302.2 Cuts	3-3
302.4 Fills.....	3-5
302.5 Setbacks.....	3-6
302.6 Drainage and Terracing	3-6
302.7 Erosion Control	3-7
SECTION 303 - FOUNDATION INVESTIGATION	3-8
303.1 General	3-8
303.2 Soil Classification	3-9
303.3 Questionable Soils.....	3-9
303.4 Liquefaction Study	3-9
303.5 Expansive Soils	3-9
303.6 Compressible Soils	3-10
303.7 Reports	3-10
303.8 Soil Tests.....	3-10
303.9 Liquefaction Potential and Soil Strength Loss	3-12
303.10 Adjacent Loads.....	3-12
303.11 Drainage	3-12
303.12 Plate Load Test.....	3-12
SECTION 304 - ALLOWABLE FOUNDATION AND LATERAL PRESSURES	3-12
304.1 From Geotechnical Site Investigation and Assessment.....	3-12
304.2 Presumptive Load-Bearing and Lateral Resisting Values	3-13
304.3 Minimum Allowable Pressures	3-13
304.4 Foundations Adjacent to Existing Retaining/Basement Walls	3-13
SECTION 305 - FOOTINGS.....	3-14
305.1 General	3-14
305.2 Footing Design	3-14
305.4 Stepped Foundations	3-14
305.5 Footings on or Adjacent to Slopes.....	3-15
305.6 Foundation Plates or Sills.....	3-16
305.7 Designs Employing Lateral Bearing	3-16
305.8 Grillage Footings.....	3-17
305.9 Bleacher Footings.....	3-17

SECTION 306 - PILES-GENERAL REQUIREMENTS	3-17
306.1 General	3-17
306.2 Interconnection	3-17
306.3 Determination of Allowable Loads.....	3-17
306.4 Static Load Test	3-18
306.5 Dynamic Load Test	3-18
306.6 Column Action	3-18
306.7 Group Action.....	3-18
306.8 Lateral Loads	3-18
306.9 Piles in Subsiding Areas	3-19
306.10 Water Jetting.....	3-19
306.10 Protection of Pile Materials	3-19
306.12 Allowable Loads.....	3-19
306.13 Use of Higher Allowable Pile Stresses	3-19
SECTION 307 - PILES-SPECIFIC REQUIREMENTS	3-20
307.1 Round Wood Piles	3-20
307.2 Uncased Cast-In-Place Concrete Piles.....	3-20
307.3 Metal-Cased Concrete Piles	3-20
307.4 Precast Concrete Piles	3-21
307.5 Precast Prestressed Concrete Piles (Pretensioned)	3-21
307.6 Structural Steel Piles.....	3-22
307.7 Concrete-Filled Steel Pipe Piles	3-22
SECTION 308 - FOUNDATION CONSTRUCTION-SEISMIC ZONE 4	3-23
308.1 General	3-23
308.2 Foundation and Geotechnical Investigations	3-23
308.3 Footing Foundations	3-23
308.4 Pier and Pile Foundations	3-23
308.5 Driven Pile Foundations	3-24
308.6 Cast-In-Place Concrete Foundations	3-26
SECTION 309 - SPECIAL FOUNDATION, SLOPE STABILIZATION AND MATERIALS OF CONSTRUCTION.....	26
309.1 Special Foundation Systems	3-26
309.2 Acceptance and Approval.....	3-26
309.3 Specific Applications.....	3-26

SECTION 301 GENERAL REQUIREMENTS

301.1 Scope

This chapter sets forth requirements for excavations, fills, footings and foundations for any building or structure.

301.2 Quality and Design

The quality and design of materials used structurally in excavations, fills, footings and foundations shall conform to the requirements specified in Chapters 4, 5, 6 and 7.

301.3 Allowable Bearing Pressures

Allowable stresses and design formulas provided in this chapter shall be used with the allowable stress design load combinations specified in Section 203.4.

301.4 Definitions

See Sections 102 and 202.

SECTION 302 EXCAVATIONS AND FILLS

302.1 General

Excavation or fills for buildings or structures shall be constructed or protected such that they do not endanger life or property. Reference is made to Section 109 of this code for requirements governing excavation, grading and earthwork construction, including fills and embankments.

302.2 Cuts

302.2.1 General

Unless otherwise recommended in the approved geotechnical engineering report or engineering report, cuts shall conform to the provisions of this section. In the absence of an approved geotechnical engineering report, these provisions may be waived for cuts 3 m or less in height, involving intact rock or hard soil, that are not intended to support structures.

302.2.2 Slopes

The slope of cut surfaces shall be no steeper than is safe for the intended use and shall be no steeper than 1 unit vertical in 2 units horizontal (50% slope) unless a geotechnical engineering report, stating that the site has been investigated, and giving an opinion that a cut at a steeper slope shall be stable and not create a hazard to public or private property, is submitted and approved. Such cuts shall be protected against erosion or degradation by sufficient cover, drainage, engineering and/or biotechnical means.

302.3 Excavations

302.3.1 Footings

Existing footings or foundations which may be undermined by any excavation shall be underpinned adequately or otherwise protected against settlement and shall be protected against lateral movement.

302.3.2 Protection of Adjoining Property

The following provisions shall apply unless prevailing local laws are deemed more stringent from an engineering standpoint:

1. Before commencing the excavation, the person making or causing the excavation to be made shall notify in writing the owners of adjoining building not less than 10 days before such excavation is to be made and that the adjoining building shall be protected. The condition of the adjoining building shall be documented to include photographs prior to excavation. Technical documents pertaining to the proposed underpinning and excavation plan shall be provided the owner of the adjacent property.

302.3.3 Support of Excavations and Open Cuts

Excavations or open cuts in excess of 1.5m in depth shall have adequately designed shoring or support to protect against collapse.

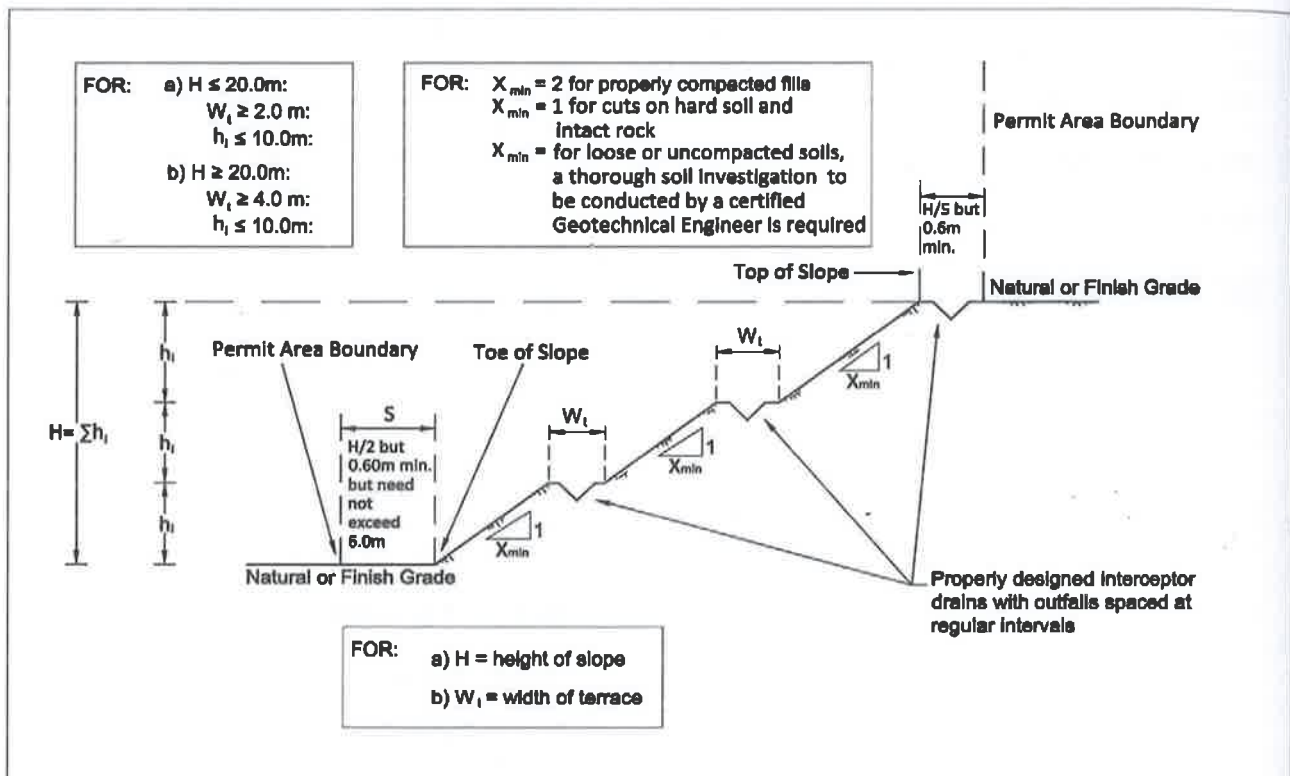


Figure 302-1 Cut Slopes

2. Unless it can be shown through a detailed geotechnical investigation that underpinning is unnecessary, any person making or causing an excavation shall protect the excavation so that the soil of adjoining property will not cave in or settle.

In cases where the adjacent existing structure will have more basements than the proposed building, the foundation of the proposed building should be designed so as not to impart additional lateral earth pressures on the existing building (see Section 304.4).

302.4 Fills

302.4.1 General

Unless otherwise recommended in the approved geotechnical engineering report, fills shall conform to the provisions of this section. In the absence of an approved geotechnical engineering report or engineering report, these provisions may be waived for minor fills ($H \leq 2.0$ m) not intended to support structures.

Fills to be used to support the foundations of any building or structure shall be placed in accordance with accepted engineering practice. A geotechnical investigation report and a report of satisfactory placement of fill, both acceptable to the building official, shall be submitted when required by the building official.

No fill or other surcharge loads shall be placed adjacent to any building or structure unless such building or structure is capable of withstanding the additional vertical and horizontal loads caused by the fill or surcharge.

Fill slopes shall not be constructed on natural slopes steeper than 1 unit vertical in 2 units horizontal (50% slope).

302.4.2 Preparation of Ground

The existing ground surface shall be adequately prepared to receive fill by removing any deleterious materials, non-complying fill, topsoil and other unsuitable materials, and by scarifying to provide a bond with the new fill.

Where the natural slopes are steeper than 1 unit vertical in 5 units horizontal (20% slope) and the height is greater than 1.5 m, the ground surface shall be prepared by benching into sound bedrock or other competent material as determined by the geotechnical engineer. The bench under the toe of a fill on a slope steeper than 1 unit vertical in 5 units horizontal (20% slope) shall be at least 3 m wide.

The area beyond the toe of fill shall be sloped to drain or a paved drain shall be provided. When fill is to be placed over a cut, the bench under the toe of fill shall be at least 3 m wide but the cut shall be made before placing the fill and acceptance by the geotechnical engineer as a suitable foundation for fill.

302.4.3 Fill Material

Any organic or deleterious material shall be removed and will not be permitted in fills. Except as permitted by the geotechnical engineer, no rock or similar irreducible material with a maximum dimension greater than 200 mm shall be buried or placed in fills.

Exception:

The placement of larger rock may be permitted when the geotechnical engineer properly devises a method of placement, and continuously inspects its placement and approves the fill stability. The following conditions shall also apply:

1. *Prior to issuance of the grading permit, potential rock disposal areas shall be delineated on the grading plan.*
2. *Rock sizes greater than 300 mm in maximum dimension shall be 3 m or more below grade, measured vertically.*
3. *Rocks shall be placed so as to assure filling of all voids with well-graded soil.*

302.4.4 Compaction

All fills shall be compacted in lifts not exceeding 20 cm in thickness to a minimum of 90 percent of maximum density as determined by ASTM Standard D-1557. In-place density shall be determined in accordance with ASTM D-1556, D-2167, D-2922, D-3017 or equivalent. For clean granular materials, the use of the foregoing procedures is inappropriate. Relative density criteria shall be used based on ASTM D5030-04. A minimum of three tests for every 500 m² area should be performed for every lift to verify compliance with compaction requirements.

302.4.5 Slope

The slope of fill surfaces shall be no steeper than is safe for the intended use. Fill slopes shall be no steeper than 1 unit vertical in 2 units horizontal (50% slope) unless substantiating data justifying steeper slopes are submitted and approved.

302.5 Setbacks

302.5.1 General

Cut and fill slopes shall be set back from site boundaries in accordance with this section subject to verification with detailed slope stability study. Setback dimensions shall be horizontal distances measured perpendicular to the site boundary. Setback dimensions shall be as shown in Figure 302-1.

302.5.2 Top of Cut Slope

The top of cut slopes shall not be made nearer to a site boundary line than one fifth of the vertical height of cut with a minimum of 0.6 m and a maximum of 3 m. The setback may need to be increased for any required interceptor drains.

302.5.3 Toe of Fill Slope

The toe of a fill slope shall be made not nearer to the site boundary line than one half the height of the slope with a minimum of 0.6 m and a maximum of 6 m. Where a fill slope is to be located near the site boundary and the adjacent off-site property is developed, special precautions shall be incorporated in the work as the building official deems necessary to protect the adjoining property from damage as a result of such grading. These precautions may include but are not limited to:

1. Additional setbacks.
2. Provision for retaining or slough walls.
3. Mechanical stabilization or chemical treatment of the fill slope surface to minimize erosion.
4. Rockfall protection
5. Provisions for the control of surface subsurface waters.

302.5.4 Modification of Slope Location

The building official may approve alternate setbacks. The building official may require an investigation and recommendation by a qualified geotechnical engineer to demonstrate that the intent of this section has been satisfied.

302.6 Drainage and Terracing

302.6.1 General

Unless otherwise indicated on the approved grading plan, drainage facilities and terracing shall conform to the provisions of this section for cut or fill slopes steeper than 1 unit vertical in 3 units horizontal (33.3% slope).

302.6.2 Terraces

Terraces at least 2.0m in width shall be established at not more than 10.0m vertical intervals on all cut or fill slopes to control surface drainage and debris except that where only one terrace is required, it shall be at mid-height. For cut or fill slopes greater than 20.0m in vertical height, terraces at least 4.0m in width shall be established at not more than 10.0m vertical intervals. Terrace widths and vertical spacing for cut and fill slopes greater than 40.0m in height shall be designed by the Geotechnical Engineer and approved by the Building Official. Suitable access shall be provided to permit proper cleaning and maintenance.

Swales or ditches on terraces shall be designed to effectively collect surface water and discharge to an outfall. It shall have a minimum gradient of 0.5 percent and must be paved with reinforced concrete not less than 75 mm in thickness or an approved equal paving material.

A single run of swale or ditch shall not collect runoff from a tributary area exceeding 1,000 m² (projected area) without discharging into a down drain.

302.6.3 Subsurface Drainage

Cut and fill slopes shall be provided with surface drainage as necessary for stability.

302.6.4 Disposal

All drainage facilities shall be designed to carry waters to the nearest practicable drainage way approved by the Building Official or other appropriate jurisdiction as a safe place to deposit such waters. Erosion of ground in the area of discharge shall be prevented by installation of non-erosive down drains or other devices or splash blocks.

Building pads shall have a drainage gradient of 2 percent toward approved drainage facilities, unless waived by the Building Official.

Note:

The gradient from the building pad may be 1 percent if all of the following conditions exist throughout the permit area:

1. *No proposed fills are greater than 3 m in maximum depth.*
2. *No proposed finish cut or fill slope faces have a vertical height in excess of 3 m.*
3. *No existing slope faces steeper than 1 unit vertical in 10 units horizontal (10% slope) have a vertical height in excess of 3m.*

302.6.5 Interceptor Drains

Paved or Lined interceptor drains shall be installed along the top of all cut slopes where the tributary drainage area above slopes toward the cut has a drainage path greater than 12 m measured horizontally. Unless specified otherwise by the Engineer of Record, interceptor drains shall be paved with a minimum of 75 mm of concrete or gunite and reinforced. They shall have a minimum depth of 300 mm and a minimum paved width of 750 mm measured horizontally across the drain. The slope of drain shall be approved by the building official. Interceptor drains shall be provided with outfalls spaced at regular intervals sufficient enough to avoid overflow of drains. In cases where neither reinforced concrete nor reinforced gunite shall be used as the paving material for the drain, sufficient testing on the alternative material must be conducted to determine its effectiveness and durability as a channel lining material.

302.7 Erosion Control

302.7.1 Slopes

The faces of cut and fill slopes shall be prepared and maintained to control against erosion. This erosion control may consist of biotechnical or geosynthetic intervention adapted to the local conditions. The protection for the slopes shall be installed as soon as practicable and prior to calling for final approval. Where cut slopes are not subject to erosion due to the erosion-resistant character of the materials, such protection may be omitted.

302.7.2 Other Devices

Where necessary, check dams, cribbing, riprap or other devices or methods shall be employed to control erosion and provide safety.

302.7.2 Scour Protection

Retaining structures and foundations located on stream banks and beds shall be provided with appropriate countermeasures, designed on the basis of a detailed engineering study, for long-term protection against scouring and erosion.

302.8 MSE Structures and Similar Reinforced Embankments and Fills

The design of Mechanically Stabilized Earth (MSE) Structures and Similar Reinforced Embankments and Fills shall incorporate provisions for internal and external drainage.

The design for the required reinforcement shall take into consideration the detrimental effects of corrosion, chemical

and biological attack, mechanical damage, creep, installation damage and pH conditions to the reinforcement.

Select Granular Backfill shall consist of sound, durable, granular material free from organic matter or other deleterious material (such as shale or other soft particles with poor durability).

The select granular backfill materials for these earth structures shall conform to Grading Requirements as stated in Table 302.1

Table 302-1 Grading Requirements

Standard Sieve Opening (mm)	Percent by Mass Passing Designated Sieve (AASHTO T 27 and T 11)
100	100
0.0425	0 – 60
0.075	0 – 15

The angle of internal friction for the backfill material shall not be less than 34°.

The assigned cohesion value during the design stage for the backfill material within the reinforced zone shall not exceed 5 kPa.

The soils should be compacted to no less than 95% MDD determine according to AASHTO T 99 Method C or D and corrected for oversized material according to AASHTO T 99, Note 9.

The material shall have a Plasticity Index of not more than 15 for rigid faced MSE structures, and not more than 20 for flexible or ductile faced MSE structures, as determined by AASHTO T 90.

Electrochemical requirements for Mechanically Stabilized Earth (MSE) retaining walls with metallic reinforcements shall comply with summarized in Table 302-2.

Table 302-2 Electrochemical Requirements

Test	Requirements
Resistivity, AASHTO T 288	3000 Ω-cm min.
pH, AASHTO T 289	5.0 to 10.0
Sulfate Content AASHTO T 290	200 ppm max.
Chloride Content AASHTO T 291	100 ppm max.

Electrochemical requirements for Mechanically Stabilized Earth MSE retaining walls with Geosynthetic

reinforcements specify that the pH shall be between 5.0 to 10.0 as determined by AASHTO T 289.

SECTION 303 FOUNDATION INVESTIGATION

303.1 General

Foundation investigation shall be conducted and a Professional Report shall be submitted at each building site. For structures two storeys or higher, an exhaustive geotechnical study shall be performed to evaluate in-situ soil parameters for foundation design and analysis. The minimum required number of boreholes per structure based on footprint area is summarized in Table 303-1. All of these boreholes should fall within the footprint of the structure, and should generally be uniformly distributed throughout the building footprint. Unless specified by the consulting Geotechnical Engineer, all boreholes should be drilled to a depth of at least five meters into hard strata or until a suitable bearing layer is reached. For buildings with basements, the depth of boring should extend to twice the least dimension of the structure's footprint (2B) added to the depth of the basement.

Table 303-1 Minimum required number of boreholes per structure.

FOOTPRINT AREA OF STRUCTURE (m^2)	MINIMUM REQUIRED NUMBER OF BOREHOLES*
$A \leq 50$	1
$50 < A \leq 500$	2
$A \geq 500$	$2 + (A/1000)**$ (Rounded Up to Nearest Integer)

*The minimum required number of boreholes should in no way be construed as an upper limit value.

** "A" corresponds to the footprint area of the structure in m^2 .

An exhaustive geotechnical investigation should also be conducted in cases of:

- 1) questionable soils, expansive soils, or problematic soils (e.g. liquefiable, organic, compressible, sensitive, etc.);

- 2) to determine whether the existing groundwater table is above or within 1.5 meters below the elevation of the lowest floor level;
- 3) where such floor is located below the finished ground level adjacent to the foundation;
- 4) in cases where the use of pile foundations and/or ground improvement are anticipated;
- 5) in areas underlain by rock strata where the rock is suspected to be of questionable characteristics or indicate variations in the structure of the rock or where solution cavities or voids are expected to be present in the rock; and
- 6) other cases deemed necessary by the Geotechnical Engineer.

The Building Official may require that the interpretation and evaluation of the results of the foundation investigation be made by a geotechnical engineer.

303.2 Soil Classification

For the purposes of this chapter, the definition and classification of soil materials for use in Table 304-1 shall be according to ASTM D-2487.

Soil classification shall be based on observation and any necessary tests of the materials disclosed by borings or excavations made in appropriate locations. Additional studies may be necessary to evaluate soil strength, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction susceptibility and expansion potential.

303.3 Questionable Soils

Where the classification, strength or compressibility of the soil are unknown, or where a load bearing value superior to that specified in this code is claimed, the Building Official shall require that these be verified through the necessary geotechnical study stipulated in Section 303.1.

303.4 Liquefaction Study

A liquefaction susceptibility assessment in accordance with accepted practice is warranted if both conditions below are discovered during the course of the geotechnical investigation:

1. Shallow ground water, 2 m or less.
2. Unconsolidated saturated sandy alluvium ($N < 15$)

Exception:

The building official may waive this evaluation upon receipt of written opinion of a qualified geotechnical engineer that liquefaction is not probable.

303.5 Expansive Soils

Soils meeting all four of the following provisions shall be considered expansive:

1. Plasticity index (PI) of 15 or greater, determined in accordance with ASTM D 4318.
2. More than 10 percent of the soil particles pass a No. 200 sieve (75 μ m), determined in accordance with ASTM D-422.
3. More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D-422.
4. Expansion index greater than 20, determined in accordance with ASTM D-4829.

Tests that show compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted.

303.5.1 Design for Expansive Soils

Footings or foundations for buildings and structures founded on expansive soils shall be designed in accordance with Section 1805.8.1 or 1805.8.2.

Footing or foundation design need not comply with Section 303.5.3 or 303.5.4 where the soil is removed in accordance with Section 303.5.4, nor where the building official approves stabilization of the soil in accordance with Section 303.5.5.

303.5.2 Foundations

Footings or foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Deflection and racking of the supported shall be limited to that which will not interfere with the usability and serviceability of the structure.

Foundations placed below where volume change occurs or below expansive soil shall comply with the following provisions:

1. Foundations extending into or penetrating expansive soils shall be designed to prevent uplift of the supported structure.
2. Foundations penetrating expansive soils shall be designed to resist forces exerted on the foundation due

to soil volume changes or shall be isolated from the expansive soil.

303.5.3 Slab-on-Ground, Foundations

Moments, shears and deflections for use in designing slab-on-ground, mat or raft foundations on expansive soils shall be determined in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations or PTI Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils*. Using the moments, shears and deflections determined above, non-prestressed slabs-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *PTI Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils*. It shall be permitted to analyze and design such slabs by other methods that account for soil-structure interaction, the deformed shape of the soil support, the plate or stiffened plate action of the slab as well as both center lift and edge lift conditions. Such alternative methods shall be rational and the basis for all aspects and parameters of the method shall be available for peer review.

303.5.4 Removal of Expansive Soils

Where expansive soil is removed in lieu of designing footings or foundations in accordance with Section 302.3.2, the soil shall be removed to a depth sufficient to ensure a constant moisture content in the remaining soil. Fill material shall not contain expansive soils and shall comply with Section 302.3.3.

Exception:

Expansive soil need not be removed to the depth of constant moisture, provided the confining pressure in the expansive soil created by the fill and supported structure exceeds the swell pressure provided that the confining pressure resulting from the fill and structural dead loads exceed the swell pressure by 20%.

303.5.5 Stabilization

Where the active zone of expansive soils is stabilized in lieu of designing footings or foundations in accordance with Section 306.2, the soil shall be stabilized by chemical, dewatering, pre-saturation or equivalent established techniques.

303.6 Compressible Soils

If the borehole data show that the proposed structures are to be built above compressible fine-grained soils (with $N < 6$), it is recommended that consolidation tests be performed in accordance with ASTM D 2435 to determine the settlement parameters for the site.

If wide, massive loads within the structures to be built on compressible fine-grained soils are to be expected for prolonged periods of time, the settlement effects on existing adjacent structures should be evaluated as well.

303.7 Reports

The soil classification and design-bearing capacity shall be shown on the plans, unless the foundation conforms to Table 305-1. The building official may require submission of a written report of the investigation, which shall include, but need not be limited to, the following information:

1. A plot showing the location of all test borings, surroundings and/or in-situ tests and excavations.
2. Technical descriptions and classifications of the materials encountered.
3. Elevation of the water table, if encountered.
4. Recommendations for foundation type and design criteria, including bearing capacity, provisions to mitigate the effects of differential settlements and expansive soils, provisions to mitigate the effects of liquefaction and soil strength loss, provisions for special foundation solutions, provisions for ground improvement measures, and effects of loads on and due to adjacent structures.
5. Expected total and differential settlement.
6. Laboratory test results of soil samples.
7. Field borehole log containing the following information
 - a. Project location
 - b. Depth of borehole
 - c. Ground elevation
 - d. Ground water table elevation
 - e. Date started and finished

The soil classification and design-bearing capacity shall be shown on the plans, unless the foundation conforms to Table 305-1.

When expansive soils are present, the Building Official may require that special provisions be made in the foundation design and construction to safeguard against damage due to this expansiveness. The building official may require a special investigation and report to provide these design and construction criteria.

303.8 Soil Tests

Tables 303-2 and 303-3 summarize the commonly used field and laboratory tests needed in determining the in-situ

soil parameters for use in foundation design and analysis.

Table 303-2 Laboratory and Field Tests

Laboratory / Field Test	ASTM/ Test Designation	Output Data / Parameter Obtained
Classification of Soils		
Moisture content	D2216-05	Moisture/ water content
Grain size analysis	D422-63	Soil gradation
Atterberg Limits	D4318-05	Liquid limit, plastic limit
USCS	D2487-00	Classification of soils
Specific Gravity	D854-05	Specific gravity
Shrinkage Limit	D427-04	Shrinkage limit
Organic Matter	D2974-00	Moisture content, ash content and percent organic matter in soil
Swedish Weight Sounding Test	JIS A1221:2002	N _{sw} -value indicating, undrained soil shear strength
UCT Test (Soils)	D2166-00	Strength parameters
Tri-axial (UU Test)	D2850-03a	Strength parameters
Tri-axial (CU Test)	D4767-04	Strength parameters
Oedometer (1-D Consolidation)	D2435-04	Consolidation parameters
Laboratory Vane Shear	D4648-05	Strength parameters
Direct Shear Test	D3080-04	Strength parameters
UCT for Intact Rock	D2938-95	Strength parameters
Standard Penetration Test	D1586-99	N-value
Modified Proctor Test	D1557-02	Maximum dry density
Standard Proctor Test	D698-00a	Maximum dry density
Field Density Test	D1556-00	Maximum dry density
CBR Lab Test	D1883-05	CBR
Cone Penetration Test	D3441-05	Soil strength parameters

Table 303-3 Geophysical Tests

Field Test	ASTM Designation	Output Data / Parameter Obtained
Geophysical Tests		
Seismic refraction	D 5777-00	Maps subsurface geologic conditions, lithologic units and fractures.
Seismic refraction	D 7128	Map the top of bedrock. Estimate elastic wave velocity of subsurface materials.
Ground Penetrating Radar (GPR)	D 6432-11	Maps lateral continuity of lithologic units and detects changes in the acoustic properties of subsurface geomaterials.
Crosshole seismic survey	D 4428	p-wave and s-wave velocity determination, elastic moduli determination
Downhole seismic survey	D 7400	p-wave and s-wave velocity determination, elastic moduli determination
Geo-resistivity Survey	D 6431-99	Determine horizontal traveling compression and shear seismic waves at test sites.

303.9 Liquefaction Potential and Soil Strength Loss

When required by Section 303.3, the potential for soil liquefaction and soil strength loss during earthquakes shall be evaluated during the geotechnical investigation. The geotechnical evaluation shall assess potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and discuss mitigating measures. Such measures shall be given consideration in the design of the building and may include, but are not limited to: ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements, or any combination of these measures.

The potential for liquefaction and soil strength loss shall be evaluated for a site peak ground acceleration that, as a minimum, conforms to the probability of exceedance specified in Section 208.6.2. Peak ground acceleration may be determined based on a site-specific study taking into account soil amplification effects.

In the absence of such a study, peak ground acceleration may be assumed equal to the seismic zone factor in Table 208-3.

303.10 Adjacent Loads

Where footings are placed at varying elevations, the effect of adjacent loads shall be included in the foundation design.

303.11 Drainage

Provisions shall be made for the control and drainage of surface water around buildings. (See also Section 305.5.5.) and ensure that scour will not threaten such structures.

303.12 Plate Load Test

The plate load test is generally used for determination of soil subgrade properties for rigid foundations. If used for building foundations, it must be emphasized that the Depth of Influence is only up to twice (2B) the width (B) of the test plate. Care must be used when extending the results to deeper depths as well as layered soils and variable subsurface conditions.

SECTION 304

ALLOWABLE FOUNDATION AND LATERAL PRESSURES

304.1 From Geotechnical Site Investigation and Assessment

The recommended allowable foundation and lateral pressures shall be estimated from a reasonably exhaustive geotechnical site investigation and assessment, which shall include at least the following:

- a. Description of regional geologic characteristics;
- b. Characterization of in-situ geotechnical conditions;
- c. Factual report on the in-situ and laboratory tests performed to characterize the site (See Section 303.7 for a list of in-situ and laboratory tests commonly carried out for geotechnical site characterization);
- d. Disclosure of the assumptions and the applicable analytical or empirical models used in estimating the allowable foundation and lateral pressures;
- e. Calculations carried out and Factor of Safety (FS) assumed in arriving at the recommended allowable foundation and lateral pressures; and
- f. Evaluation of existing potential geologic hazards and those that may be induced or triggered by the construction/installation of the structure.

The geotechnical site investigation and assessment shall be performed by a geotechnical engineer.

A geotechnical investigation and assessment shall be presented in a report. The report, together with a brief resume and a sworn statement of accountability of the geotechnical engineering consultant who prepared it, shall be included in the submittals to be reviewed and examined by the building official or government authority in charge of issuing the relevant permits such as environmental compliance certificate and/or building permit.

304.2 Presumptive Load-Bearing and Lateral Resisting Values

When no exhaustive geotechnical site assessment and investigation is performed, especially when no in-situ or very limited tests are carried out, the presumptive load-bearing and lateral resisting values provided in **Table 304-1** shall be used. Use of these values requires that the foundation design engineer has, at the least, carried out an inspection of the site and has become familiar with the predominant soil or rock characteristics of the site.

Presumptive load-bearing values shall apply to materials with similar physical characteristics and dispositions. Mud, organic silt, organic clays, peat or unprepared fill shall not be assumed to have a presumptive load-bearing capacity unless data from a geotechnical site assessment and investigation to substantiate the use of such a value are submitted.

For clay, sandy clay, silty clay and clayey silt, in no case shall the lateral sliding resistance exceed one-half the dead load.

304.3 Minimum Allowable Pressures

The recommended allowable foundation and lateral values shall be with the allowable stress design load combinations specified in Section 203.4.

304.4 Foundations Adjacent to Existing Retaining/Basement Walls

In cases where the adjacent building will have more basements than the proposed building, the foundation of the proposed building should be designed so as not to impart additional lateral earth pressures on the existing building.

Table 304-1 Allowable Foundation and Lateral Pressure

Class of Materials ¹	Allowable Foundation Pressure ² (kPa)	Lateral Bearing Below Natural Grade ³ (kPa/m of depth)	Lateral Sliding ⁴	
			Coefficient ⁵	Resistance ⁶ (kPa)
1. "Intact" Tuffaceous Sandstone ^a	1,000	300	-	-
2. "Lightly Weathered" Tuffaceous Sandstone ^b	500	150	-	-
3. Sandy Gravel and /or Gravel(GW & GP)	100	30	0.35	-
4. Well-graded Sand, Poorly-graded Sand, Silty Sand, Clayey Sand, Silty Gravel and Clayey Gravel (SW, SP, SM, SC, GM and GC)	75	25	0.25	-
5. Clay, Sandy Clay, Silty Clay and Clayey Silt (CL, ML, MH, and CH)	50 ^c	15	-	7

¹ A geotechnical site investigation is recommended for soil classification (Refer to Section 303).

² All values of allowable foundation pressure are for footings having a minimum width of 300mm and a minimum depth of 300mm into the natural grade. Except as noted in Footnote 'a', an increase of 20% is allowed for each additional 300mm of width and/or depth to a maximum value of three times the designated value. An increase of one-third is permitted when using the alternate load combinations in Section 203.4 that include wind or earthquake loads.

³ The resistance values derived from the table are permitted to be increased by the tabular value for each additional 300 mm of depth to a maximum of 15 times the tabular value. Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a 12mm motion at the ground surface due to short-term lateral loads are permitted to be designed using lateral-bearing values equal to two times the tabular values.

⁴ Lateral bearing and sliding resistance may be combined.

⁵ Coefficient to be multiplied by the dead load.

⁶ Lateral sliding resistance value to be multiplied by the contact area. In no case shall the lateral sliding resistance exceed one-half the dead load.

^a Must satisfy both $UCT_{min} = 3\text{Mpa}$ and $RQD \geq 70$

^b Must satisfy both $UCT_{min} = 1\text{Mpa}$ and $RQD \geq 50$

^c No increase shall be allowed for an increase of width.

SECTION 305 FOOTINGS

305.1 General

Footings and foundations shall be constructed of masonry, concrete or treated wood in conformance with Chapters 4, 6 and 7. Footings of concrete and masonry shall be of solid material. Foundations supporting wood shall extend at least 150 mm above the adjacent finish grade. Footings shall have a minimum depth as indicated in Table 305-1, unless another depth is warranted, as established by a foundation investigation.

The provisions of this section do not apply to building and foundation systems in those areas subject to scour and water pressure by wind and wave action. Buildings and foundations subject to such loads shall be designed in accordance with approved national standard

Table 305-1 Minimum Requirements for Foundations ^{1,2,3}

Number of Floors Supported by the Foundations	Thickness of Foundation Wall (mm)		Width of Footing (mm)	Thickness of Footing (mm)	Depth Below Undisturbed Ground Surface (mm) ⁴
	Concrete	Unit Masonry			
1	150	150	300	150	300
2	200	200	375	175	450
3	250	250	450	200	600

¹ Where unusual conditions are found, footings and foundations shall be as required in Section 305.1.

² The ground under the floor may be excavated to the elevation of the top of the footing.

³ Foundation may support a roof in addition to the stipulated number of floors. Foundations supporting roofs only shall be as required for supporting one floor.

305.2 Footing Design

Except for special provisions of Section 307 covering the design of piles, all portions of footings shall be designed in accordance with the structural provisions of this code and shall be designed to minimize differential settlement when necessary and the effects of expansive soils when present.

Slab-on-grade and mat-type footings for buildings located on expansive soils may be designed in accordance with the geotechnical recommendation as permitted by the building official.

305.2.1 Design Loads

Footings shall be designed for the most unfavorable load effects due to combinations of loads. The dead load is permitted to include the weight of foundations, footings and overlying fill. Reduced live loads as permitted in the Chapter on Loadings are permitted to be used in the design of footings.

305.2.2 Vibratory Loads

Where machinery operations or other vibratory loads or vibrations are transmitted to the foundations, consideration shall be given in the Report to address the foundation design to prevent detrimental disturbances to the soil.

305.3 Bearing Walls

Bearing walls shall be supported on masonry or concrete foundations or piles or other permitted foundation system that shall be of sufficient size to support all loads.

Where a design is not provided, the minimum foundation requirements for stud bearing walls shall be as set forth in Table 305-1, unless expansive soils of a severity to cause differential movement are known to exist.

Exceptions:

1. A one-story wood or metal-frame building not used for human occupancy and not over 40 m² in floor area may be constructed with walls supported on a wood foundation plate permanently under the water table when permitted by the building official.
2. The support of buildings by posts embedded in earth shall be designed as specified in Section 305.7. Wood posts or poles embedded in earth shall be pressure treated with an approved preservative. Steel posts or poles shall be protected as specified in Section 306.10.

305.4 Stepped Foundations

Foundations for all buildings where the surface of the ground slopes more than 1 unit vertical in 10 units horizontal (10% slope) shall be level or shall be stepped so that both top and bottom of such foundation are level.

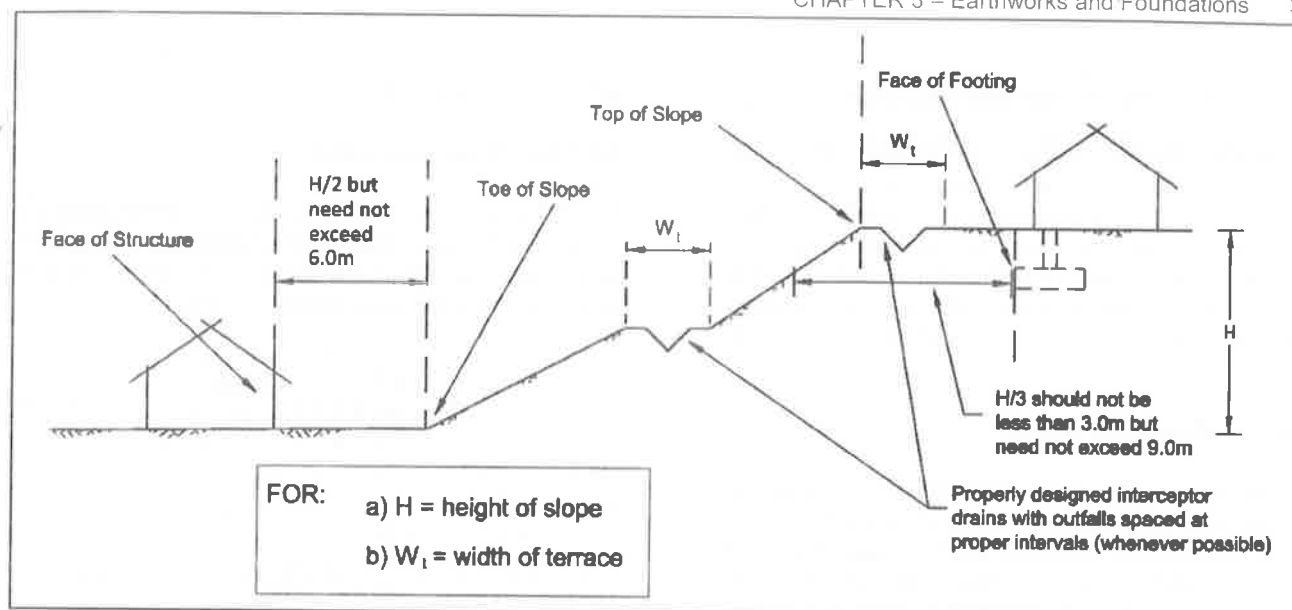


Figure 305-1 Setback Dimensions for Building Clearance for Stable Natural Slopes on Firm and Intact Ground

305.5 Footings on or Adjacent to Slopes

305.5.1 Scope

The placement of buildings and structures on or adjacent to slopes steeper than 1 unit vertical in 3 units horizontal (33.3% slope) shall be in accordance with this section.

305.5.2 Building Clearance from Ascending Slopes

In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided for in Section 305.5.6 and Figure 305-1, the following criteria will be assumed to provide this protection. Where the existing slope is steeper than 1 unit vertical in 1 unit horizontal (100% slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.

305.5.3 Footing Setback from Descending Slope Surface

Footings on or adjacent to slope surfaces shall be founded in firm material with an embedment and setback from the slope surface sufficient to provide vertical and lateral support for the footing without detrimental settlement. Except as provided for in Section 305.5.6 and Figure 305-1, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than 1 unit vertical in 1 unit horizontal (100% slope), the required setback shall be measured from an imaginary plane 45 degrees to the horizontal, projected upward from the toe of the slope.

305.5.4 Pools

The setback between pools regulated by this code and slopes shall be equal to one half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 2 meters from the top of the slope shall be capable of supporting the water in the pool without soil support.

305.5.5 Foundation Elevation

On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 300 mm plus 2 percent. The building official may permit alternate elevations, provided it can be demonstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.

305.5.6 Alternate Setback and Clearance

The Building Official may approve alternate setbacks and clearances. The building official may require an investigation and recommendation of a qualified engineer to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

305.5.7 Reinforced Slopes

Footings on or adjacent to slopes reinforced using rock anchors, soils nails, geosynthetics or any other similar ground improvement technique shall be founded such that it does not interfere with or impair the function of the reinforcing elements. The Building Official may require the recommendation of a geotechnical engineer to demonstrate that the reinforced slope can safely carry the structural loads it will be subjected to.

305.6 Foundation Plates or Sills

Wood plates or sills shall be bolted to the foundation or foundation wall. Steel bolts with a minimum nominal diameter of 12 mm shall be used in Seismic Zone 2. Steel bolts with a minimum nominal diameter of 16 mm shall be used in Seismic Zone 4. Bolts shall be embedded at least 180 mm into the concrete or masonry and shall be spaced not more than 2 meters apart. There shall be a minimum of two bolts per piece with one bolt located not more than 300 mm or less than seven bolt diameters from each end of the piece. A properly sized nut and washer shall be tightened on each bolt to the plate. Foundation plates and sills shall be the kind of wood specified in Chapter 6.

305.7 Designs Employing Lateral Bearing**305.7.1 General**

Construction employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth may be used and designed to resist both axial and lateral loads. The depth to resist lateral loads shall be determined by means of the design criteria established herein or other methods approved by the building official.

305.7.2 Design Criteria**305.7.2.1 Non-constrained**

The following formula may be used in determining the depth of embedment required to resist lateral loads where no constraint is provided at the ground surface, such as rigid floor or rigid ground surface pavement.

$$d = \frac{A}{2} \left(1 + \sqrt{1 + \frac{4.3h}{A}} \right) \quad (305-1)$$

where:

$$A = \frac{2.3P}{S_1 b}$$

- b = diameter of round post or footing or diagonal dimension of square post or footing, m.
- d = depth of embedment in earth in m but not over 3.5 m for purpose of computing lateral pressure, m
- h = distance from ground surface to point of application of P , m
- P = applied lateral force, kN
- S_1 = allowable lateral soil-bearing pressure as set forth in Table 304-1 based on a depth of one third the depth of embedment, kPa
- S_3 = allowable lateral soil-bearing pressure as set forth in Table 304-1 based on a depth equal to the depth of embedment, kPa

305.7.2.2 Constrained

The following formula may be used to determine the depth of embedment required to resist lateral loads where constraint is provided at the ground surface, such as a rigid floor or pavement.

$$d^2 = 4.25 \frac{Ph}{S_3 b} \quad (305-2)$$

305.7.2.3 Vertical load

The resistance to vertical loads is determined by the allowable soil-bearing pressure set forth in Table 304-1.

305.7.3 Backfill

The backfill in the annular space around column not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with an ultimate strength of 15 MPa at 28 days. The hole shall not be less than 100 mm larger than the diameter of the column at its bottom or 100 mm larger than the diagonal dimension of a square or rectangular column.
2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 200 mm in thickness.

305.7.4 Limitations

The design procedure outlined in this section shall be subject to the following limitations:

305.7.4.1 The frictional resistance for retaining walls and slabs on silts and clays shall be limited to one half of the normal force imposed on the soil by the weight of the footing or slab.

305.7.4.2 Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided.

305.8 Grillage Footings

When grillage footings of structural steel shapes are used on soils, they shall be completely embedded in concrete. Concrete cover shall be at least 150 mm on the bottom and at least 100 mm at all other points.

305.9 Bleacher Footings

Footings for open-air seating facilities shall comply with Chapter 3.

Exceptions:

Temporary open-air portable bleachers may be supported upon wood sills or steel plates placed directly upon the ground surface, provided soil pressure does not exceed 50 kPa.

SECTION 306

PILES-GENERAL REQUIREMENTS

306.1 General

Pile foundations shall be designed and installed on the basis of a foundation investigation as defined in Section 303 where required by the building official.

The investigation and report provisions of Section 303 shall be expanded to include, but not be limited to, the following:

1. Recommended pile types and installed allowable axial capacities, and estimated settlement.
2. Driving criteria.
3. Installation procedures.
4. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).
5. Pile load test requirements.

The use of piles not specifically mentioned in this chapter shall be permitted, subject to the approval of the building official upon submission of acceptable test data, calculations or other information relating to the properties and load-carrying capacities of such piles.

306.2 Interconnection

Individual pile caps and caissons of every structure subjected to seismic forces shall be interconnected by ties. Such ties shall be capable of resisting, in tension or compression, a minimum horizontal force equal to 10 percent of the largest column vertical load.

Exception:

Other approved methods may be used where it can be demonstrated that equivalent restraint can be provided.

306.3 Determination of Allowable Loads

The allowable axial and lateral loads on piles shall be determined by an approved formula, by a foundation investigation or by load tests. Static axial compressive pile load test shall be in accordance with ASTM Standard D-1143, and lateral load testing of piles shall conform with ASTM Standard D-3966. Dynamic pile tests shall be in accordance with ASTM Standard D-4945. Static axial tensile load testing to determine the uplift capacity of pile-soil systems shall be in accordance with ASTM Standard D-3689.

306.4 Static Load Test

Static axial compressive pile load test shall be in accordance with ASTM Standard D-1143. The building official may require that the test be conducted under the supervision of a geotechnical engineer experienced and knowledgeable in the practice of static pile load testing

When the allowable axial compressive load of a single pile is determined by a static load test, one of the following methods shall be used:

Method 1. It shall not exceed 50 percent of the yield point under test load. The yield point shall be defined as that point at which an increase in load produces a disproportionate increase in settlement.

Method 2. It shall not exceed one half of the load, which causes a net settlement, after deducting rebound, of 0.03 mm/kN of test load, which has been applied for a period of at least 24 hours.

Method 3. It shall not exceed one half of that load under which, during a 40-hour period of continuous load application, no additional settlement takes place.

306.5 Dynamic Load Test

High-strain dynamic load test may be used to determine the bearing capacity of piles, in accordance with ASTM Standard D-4945. The building official may require that the test be conducted by a geotechnical engineer experienced and knowledgeable in the practice of dynamic load testing.

306.6 Column Action

All piles standing unbraced in air, water or material not capable of lateral support shall conform with the applicable column formula as specified in this code. Such piles driven into firm ground may be considered fixed and laterally supported at 1.5 m below the ground surface and in soft material at 3 m from the ground surface unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

306.7 Group Action

Consideration shall be given to the reduction of allowable pile load when piles are placed in groups.

Where soil conditions make such load reductions advisable or necessary, the allowable axial and lateral loads determined for a single pile shall be reduced by any rational method or formula submitted to the building official.

306.8 Lateral Loads

The design of piles subjected to lateral loads shall be consistent with the design rules given Sections 306.1 through 306.7, where applicable. For foundations involving piles subjected to lateral, the investigation and report provisions of Section 303 shall be expanded to include, but not be limited to:

1. Allowable lateral load capacity of recommended pile type.
2. Resulting lateral displacements at allowable lateral load.
3. Lateral pile load test requirements.

306.8.1 The design resistance of piles subjected to lateral loads, should be assessed based on one of the following failure mechanisms:

1. For short piles, rotation or translation as a rigid body
2. For long slender piles, bending failure of the pile, accompanied by local yielding and displacement of the soil near the top of the pile.

306.8.2 Pile Groups

The group effects shall be considered when assessing the resistance of laterally loaded pile groups.

306.8.3 Group interaction effects as well as head fixity shall be accounted for when deriving the lateral resistance of pile groups from results of load tests performed on individual piles.

306.8.4 When assessing lateral load resistance from results of subsurface investigation and pile strength parameters, the transverse resistance of a pile or pile group shall be calculated using a compatible set of structural effects of actions, ground reactions and displacements, and consider the possibility of structural failure of the pile in the ground as well as the degree of freedom of rotation of piles at the connection with the structure.

306.9 Piles in Subsiding Areas

Where piles are driven through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces, which may be imposed on the piles by the subsiding upper strata.

Where the influence of subsiding fills is considered as imposing loads on the pile, the allowable stresses specified in this chapter may be increased if satisfactory substantiating data are submitted.

306.10 Water Jetting

Installation of Piles by water shall not be used except where and as specifically permitted by the building official. When used, jetting shall be carried out in such a manner that the carrying capacity of existing piles and structures shall not be impaired. After withdrawal of the jet, piles shall be driven down until the required resistance is obtained.

306.10 Protection of Pile Materials

Where the boring records of site conditions indicate possible deleterious action on pile materials because of soil constituents, changing water levels or other factors, such materials shall be adequately protected by methods or processes approved by the geotechnical engineer.

The effectiveness of such methods or processes for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence, which demonstrates the effectiveness of such protective measures.

306.12 Allowable Loads

The allowable loads based on soil conditions shall be established in accordance with Section 306.

Exception:

Any uncased cast-in-place pile may be assumed to develop a frictional resistance equal to one sixth of the bearing value of the soil material at minimum depth as set forth in Table 305-1 but not to exceed 25 kPa unless a greater value is allowed by the building official after a foundation investigation as specified in Section 303 is submitted. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended after a foundation investigation as specified in Section 303.

306.13 Use of Higher Allowable Pile Stresses

Allowable compressive stresses greater than those specified in Section 307 shall be permitted when substantiating data justifying such higher stresses are submitted to and approved by the building official. Such substantiating data shall be included in the foundation investigation report in accordance with Section 306.1.

SECTION 307

PILES-SPECIFIC REQUIREMENTS

307.1 Round Wood Piles

307.1.1 Material

Except where untreated piles are permitted, wood piles shall be pressure treated. Untreated piles may be used only when it has been established that the cutoff will be below lowest groundwater level assumed to exist during the life of the structure.

307.1.2 Allowable Stresses

The allowable unit stresses for round woodpiles shall not exceed those set forth in Chapter 6.

The allowable values listed in, for compression parallel to the grain at extreme fiber in bending are based on load sharing as occurs in a pile cluster. For piles which support their own specific load, a safety factor of 1.25 shall be applied to compression parallel to the grain values and 1.30 to extreme fiber in bending values.

307.2 Uncased Cast-In-Place Concrete Piles

307.2.1 Material

Concrete piles cast in place against earth in drilled or bored holes shall be made in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft.

The length of such pile shall be limited to not more than 30 times the average diameter. Concrete shall have a specified compressive strength f'_c of not less than 17.5 MPa.

Exception:

The length of pile may exceed 30 times the diameter provided the design and installation of the pile foundation is in accordance with an approved foundation investigation report.

307.2.2 Allowable Stresses

The allowable compressive stress in the concrete shall not exceed $0.33f'_c$. The allowable compressive stress of reinforcement shall not exceed 34 percent of the yield strength of the steel or 175 MPa.

307.3 Metal-Cased Concrete Piles

307.3.1 Material

Concrete used in metal-cased concrete piles shall have a specified compressive strength f'_c of not less than 17.5 MPa.

307.3.2 Installation

Every metal casing for a concrete pile shall have a sealed tip with a diameter of not less than 200 mm.

Concrete piles cast in place in metal shells shall have shells driven for their full length in contact with the surrounding soil and left permanently in place. The shells shall be sufficiently strong to resist collapse and sufficiently watertight to exclude water and foreign material during the placing of concrete.

Piles shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. No pile shall be driven within four and one-half average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the geotechnical engineer.

307.3.3 Allowable Stresses

Allowable stresses shall not exceed the values specified in Section 307.2.2, except that the allowable concrete stress may be increased to a maximum value of $0.40f'_c$ for that portion of the pile meeting the following conditions:

1. The thickness of the metal casing is not less than 1.7 mm (No. 14 carbon sheet steel gage).
2. The casing is seamless or is provided with seams of equal strength and is of a configuration that will provide confinement to the cast-in-place concrete.
3. The specified compressive strength f'_c shall not exceed 35 MPa and the ratio of steel minimum specified yield strength F_y to concrete specified compressive strength f'_c shall not be less than 6.
4. The pile diameter is not greater than 400 mm.

307.4 Precast Concrete Piles

✓ 307.4.1 Materials

Precast concrete piles shall have a specified compressive strength f'_c of not less than 20 MPa, and shall develop a compressive strength of not less than 20 MPa before driving.

307.4.2 Reinforcement Ties

The longitudinal reinforcement in driven precast concrete piles shall be laterally tied with steel ties or wire spirals. Ties and spirals shall not be spaced more than 75 mm apart, center to center, for a distance of 600 mm from the ends and not more than 200 mm elsewhere. The gage of ties and spirals shall be as follows:

1. For piles having a diameter of 400 mm or less, wire shall not be smaller than 5.5 mm (No. 5 B.W.gage).
2. For piles, having a diameter of more than 400 mm and less than 500 mm, wire shall not be smaller than 6 mm (No.4 B.W.gage).
3. For piles having a diameter of 500 mm and larger, wire shall not be smaller than 6.5 mm (No.3 B.W. gage).

307.4.3 Allowable Stresses

Precast concrete piling shall be designed to resist stresses induced by handling and driving as well as by loads. The allowable stresses shall not exceed the values specified in Section 307.2.2.

307.5 Precast Prestressed Concrete Piles (Pretensioned)

307.5.1 Materials

Precast prestressed concrete piles shall have a specified compressive strength f'_c of not less than 35 MPa and shall develop a compressive strength of not less than 27 MPa before driving.

307.5.2 Reinforcement

307.5.2.1 Longitudinal Reinforcement

The longitudinal reinforcement shall be high-tensile seven-wire strand conforming to ASTM Standards. Longitudinal reinforcement shall be laterally tied with steel ties or wire spirals.

307.5.2.2 Transverse Reinforcement

Ties or spiral reinforcement shall not be spaced more than 75 mm apart, center to center, for a distance of 600 mm from the ends and not more than 200 mm elsewhere.

At each end of the pile, the first five ties or spirals shall be spaced 25 mm center to center.

For piles having a diameter of 600 mm or less, wire shall not be smaller than 5.5 mm (No. 5 B.W.gage).

For piles having a diameter greater than 600 mm but less than 900 mm, wire shall not be smaller than 6 mm (No. 4 B.W.gage).

For piles having a diameter greater than 900 mm, wire shall not be smaller than 6.5 mm (No.3 B.W.gage).

307.5.3 Allowable Stresses

Precast prestressed piling shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 2.5 MPa for piles up to 10 m in length, 4 MPa for piles up to 15 m in length, and 5 MPa for piles greater than 15 meters in length.

The compressive stress in the concrete due to externally applied load shall not exceed:

$$f_c = 0.33f'_c - 0.27f_{pc} \quad (307-1)$$

where

f_{pc} = effective prestress stress on the gross section.

Effective prestress shall be based on an assumed loss of 200 MPa in the prestressing steel. The allowable stress in the prestressing steel shall not exceed the values specified in Section 418.6.

307.5.4 Splicing

Where required, splicing for concrete piles shall be by use of embedded and properly anchored thick steel plates at the ends being joined which shall then be fully welded, or by use of adequate sized dowel rods and steel receiving sleeves. The dowels and the faces shall then be joined by structural epoxy. Metal splice cans are not allowed.

307.6 Structural Steel Piles**307.6.1 Material**

Structural steel piles, steel pipe piles and fully welded steel piles fabricated from plates shall conform to one of the material specifications listed in Section 501.3.

307.6.2 Allowable Stresses

The allowable axial stresses shall not exceed 0.35 of the minimum specified yield strength F_y or 85 MPa, whichever is less.

Exception:

When justified in accordance with Section 306.12, the allowable axial stress may be increased above 85 MPa and $0.35F_y$, but shall not exceed $0.5F_y$.

307.6.3 Minimum Dimensions

Sections of driven H-piles shall comply with the following:

1. The flange projection shall not exceed 14 times the minimum thickness of metal in either the flange or the web, and the flange widths shall not be less than 80 percent of the depth of the section.
2. The nominal depth in the direction of the web shall not be less than 200 mm.
3. Flanges and webs shall have a minimum nominal thickness of 10 mm.

Sections of driven pipe piles shall have an outside diameter of not less than 250 mm and a minimum thickness of not less than 6 mm.

307.7 Concrete-Filled Steel Pipe Piles**307.7.1 Material**

The steel pipe of concrete-filled steel pipe piles shall conform to one of the material specifications listed in Section 501.3. The concrete in concrete-filled steel pipe piles shall have a specified compressive strength f'_c of not less than 17.5 MPa.

307.7.2 Allowable Stresses

The allowable axial stresses shall not exceed 0.35 of the minimum specified yield strength F_y of the steel or 0.33 of the specified compressive strength f'_c of concrete, provided F_y shall not be assumed greater than 250 MPa for computational purposes.

Exception:

When justified in accordance with Section 306.12, the allowable stresses may be increased to $0.50 F_y$.

307.7.3 Minimum Dimensions

Driven piles of uniform section shall have a nominal outside diameter of not less than 200 mm.

SECTION 308 FOUNDATION CONSTRUCTION- SEISMIC ZONE 4

308.1 General

In Seismic Zones 4, the further requirements of this section shall apply to the design and construction of foundations, foundation components and the connection of superstructure elements thereto. See Section 421.10 for additional requirements for structural concrete foundations resisting seismic forces.

308.2 Foundation and Geotechnical Investigations

Where a structure is determined to be in Seismic Zone 4 in accordance with Section 208.4, an investigation shall be conducted and shall include an evaluation of the following potential hazards resulting from earthquake motions: slope instability, liquefaction and surface rupture due to faulting or lateral spreading.

In addition, the following investigations shall also be met:

1. A determination of lateral pressures on basement and retaining walls due to earthquake motions.
2. An assessment of potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall address mitigation measures. Such measures shall be given consideration in the design of the structure and can include but are not limited to ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements or any combination of these measures. The potential for liquefaction and soil strength loss shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration shall be determined from a site-specific study taking into account soil amplification effects, as specified in Section 208.4.

308.3 Footing Foundations

Where a structure is assigned to Seismic Zone 4 in accordance with Section 208.4, individual spread footings founded on soil defined in Section 208.4.3 as Soil profile Type S_E or S_F shall be interconnected by ties. Ties shall be capable of carrying, in tension or compression, unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.

308.4 Pier and Pile Foundations

Where a structure is assigned to Seismic Zone 4 in accordance with Section 208.4, the following shall apply. Individual pile caps, piers or piles shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade, reinforced concrete slabs on grade, confinement by competent rock, hard cohesive soils or very dense granular soils. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exception:

Piers supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, lightly loaded exterior decks and patios and occupancy category IV and V specified in Section 103 not exceeding two stories of light-frame construction, are not subject to interconnection if it can be shown the soils are of adequate stiffness; subject to the approval of the building official.

308.4.1 Connection to Pile Cap

For piles required to resist uplift forces or provide rotational restraint, design of anchorage of piles into the pile cap shall be provided considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the pile in tension. Anchorage into the pile cap shall be capable of developing the following:

1. In the case of uplift, the lesser of the nominal tensile strength of the longitudinal reinforcement in a concrete pile, or the nominal tensile strength of a steel pile, or the pile uplift soil nominal strength factored by 1.3 or the axial tension force resulting from the load combinations of Section 203.
2. In the case of rotational restraint, the lesser of the axial and shear forces, and moments resulting from the load combinations of Section 203 or development of the full axial, bending and shear nominal strength of the pile.

308.4.2 Design Details for Piers, Piles and Grade Beams

Piers or piles shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces. Concrete piers or piles on soil type S_E or S_F sites, as determined in Section 208.4.3, shall be designed and detailed in accordance with Sections 410 within seven pile diameters of the pile cap and the interfaces of soft to medium stiff clay or liquefiable strata. Grade beams shall be designed as beams in accordance Section 4. When grade beams have the capacity to resist the forces from the load combinations in Section 203.

308.4.3 Flexural Strength

Where the vertical lateral-force-resisting elements are columns, the grade beam or pile cap flexural strengths shall exceed the column flexural strength. The connection between batter piles and grade beams or pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be capable of resisting forces and moments from the load combinations of Section 203.

308.5 Driven Pile Foundations**308.5.1 Precast Concrete Piles**

Where a structure is assigned to Seismic Zone 4, a minimum longitudinal steel reinforcement ratio of 0.01 shall be provided for precast non-prestressed concrete piles. The longitudinal reinforcing shall be confined with closed ties or equivalent spirals of a minimum 3/8 in (10 mm) diameter. Transverse confinement reinforcing shall be provided at a maximum spacing of eight times the diameter of the smallest longitudinal bar, but not to exceed 6 in. (152 mm), within three pile diameters of the bottom of the pile cap. Outside of the confinement region, closed ties or equivalent spirals shall be provided at a 16 longitudinal-bar-diameter maximum spacing, but not greater than 8 in. (200 mm). Reinforcement shall be full length.

For Site Classes D through F, Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.6.4.2 through 21.6.4.4 of ACI 318 for the full length of the pile.

In other than Site Classes E or F, the specified transverse confinement reinforcement shall be provided within three pile diameters below the bottom of the pile cap, but it is permitted to use a transverse reinforcing ratio of not less than one-half of that required in Section 21.6.4.4(a) of ACI 318 throughout the remainder of the pile length.

308.5.2 Precast Prestressed Piles

Where a structure is assigned to Seismic Zone 4, the following shall apply:

For the upper 20 ft (6 m) of precast prestressed piles, the minimum volumetric ratio of spiral reinforcement shall not be less than 0.007 or the amount required by the following equation:

$$\rho_s = 0.12 f'_c / f_{yh} \quad (308.5.1)$$

where

- f'_c = Specified compressive strength of concrete, MPa
- f_{yh} = Yield strength of spiral reinforcement, 586 MPa
- ρ_s = Spiral reinforcement index (volume spiral/volume of core)

A minimum of one-half of the volumetric ratio of spiral reinforcement required by Eq. 308.5.1 shall be provided for the remaining length of the pile.

Requirements of ACI 318, Chapter 21, need not apply.

Where the total pile length in the soil is 35 ft (10,668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 ft (10,668 mm), the ductile pile region shall be taken as the greater of 35 ft (10,668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.

In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longitudinal strand, or 8 in (203 mm), whichever is smaller.

Spiral reinforcement shall be spliced by lapping one full turn, by welding, or by the use of a mechanical connector. Where spiral reinforcement is lap spliced, the ends of the spiral shall terminate in a seismic hook in accordance with ACI 318, except that the bend shall be not less than 135°.

Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

$$\rho_s = 0.25 \frac{f'_c}{f_{yh}} \left(\frac{A_g}{A_{ch}} - 1 \right) \left[\frac{1}{2} + \frac{1.4P}{f'_c A_g} \right] \quad (308.5.2)$$

but not less than:

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \left[\frac{1}{2} + \frac{1.4P}{f'_c A_g} \right] \quad (308.5.3)$$

and need not exceed:

$$\rho_s = 0.021 \quad (308.5.4)$$

where

- A_g = Pile cross-sectional area, mm²
- A_{ch} = Core area defined by spiral outside diameter, mm²
- f'_c = Specified compressive strength of concrete, MPa ≤ 41.4 MPa
- f_{yh} = Yield strength of spiral reinforcement ≤ 586 MPa
- P = Axial load on pile resulting from the load combination 1.2D + 0.5L + 1.0E, kN
- ρ_s = Volumetric ratio (volume of spiral/ volume of core)

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.

When transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacings, and perpendicular to dimension, h_c , shall conform to:

$$A_{sh} = 0.3sh_c \left(\frac{f'_c}{f_{yh}} \right) \left(\frac{A_g}{A_{ch}} - 1 \right) \left[\frac{1}{2} - 1.4 \frac{P}{f'_c A_g} \right] \quad (308.5.5)$$

but not less than:

$$A_{sh} = 0.12sh_c \left(\frac{f'_c}{f_{yh}} \right) \left[\frac{1}{2} + \frac{1.4P}{f'_c A_g} \right] \quad (308.5.6)$$

where

- f_{yh} = ≤ 483 MPa
- h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, mm
- s = Spacing of transverse reinforcement measured along length of pile, mm
- P = Axial load, N
- A_{sh} = Cross-sectional area of transverse reinforcement, mm²
- A_g = Gross area of pile, mm²
- f'_c = Specified compressive strength of concrete, MPa

The hoops and cross ties shall be equivalent to deformed bars not less than 10 mm in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided.

308.6 Cast-In-Place Concrete Foundations

Where a structure is assigned to Seismic Zone 4 a minimum longitudinal reinforcement ratio of 0.005 shall be provided for uncased cast-in-place drilled or augered concrete piles, piers or caissons in the top one-half of the pile length a minimum length of 10 feet (3,048 mm) below ground or throughout the flexural length of the pile, whichever length is greatest. The flexural length shall be taken as the length of the pile to a point where the concrete section cracking moment strength multiplied by 0.4 exceeds the required moment strength at that point. There shall be a minimum of four longitudinal bars with transverse confinement reinforcement provided in the pile within three times the least pile dimension of the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 410 for other than Soil Profile Type S_E , S_F or as determined in Section 208.4.3 or liquefiable sites is permitted. Tie spacing throughout the remainder of the concrete section shall neither exceed 12-longitudinal-bar diameters, one-half the least dimension of the section, nor 12 inches (305 mm). Ties shall be a minimum of 10mm bars for piles with a least dimension up to 20 inches (508 mm), and 12 mm bars for larger piles.

SECTION 309 SPECIAL FOUNDATION, SLOPE STABILIZATION AND MATERIALS OF CONSTRUCTION

309.1 Special Foundation Systems

Special foundation systems or materials other than specified in the foregoing Sections may be introduced provided that such systems can be supported by calculations and theory to be providing safe foundation systems and when approved by the Engineer of record. Materials for incorporation into the foundation should have proven track record of successful usage in similar applications.

309.2 Acceptance and Approval

Structure support on improved ground using such special systems or proprietary systems may be approved subject to submittal of calculations and other proof of acceptance and successful usage.

309.3 Specific Applications

Specialty foundation systems may be applied or used specifically to address any or combinations of the following: Bearing Capacity Improvement, Liquefaction mitigation, slope stability enhancement, control and/or acceleration of Consolidation settlements or immediate settlements, increase in soil shear capacity, increased pullout or overturning capacity, special anchors in soil and rock and other beneficial effects. Controlled low strength materials (CLSM) to reduce fill loads may be allowed for use where applicable.

Chapter 4

STRUCTURAL CONCRETE

NATIONAL STRUCTURAL CODE OF THE PHILIPPINES VOLUME I BUILDINGS, TOWERS AND OTHER VERTICAL STRUCTURES

SEVENTH EDITION, 2015

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Table of Contents

SECTION 401	7
GENERAL REQUIREMENTS.....	7
401.1 Scope	7
401.2 General	7
401.3 Purpose	7
401.4 Applicability	7
401.5 Interpretation	8
401.6 Building Official	8
401.7 Licensed Design Professional	8
401.8 Construction Documents and Design Records	9
401.9 Testing and Inspection.....	9
401.10 Approval of Special Systems of Design, Construction, or Alternative Construction Materials.....	9
401.11 Provisions for Earthquake Resistance	9
SECTION 402	9
NOTATION AND TERMINOLOGY.....	9
402.1 Scope	9
402.2 Notation	9
402.3 Terminology	20
SECTION 403	28
REFERENCED STANDARDS	28
403.1 Scope	28
403.2 Referenced Standards	28
SECTION 404	31
STRUCTURAL SYSTEM REQUIREMENTS.....	31
404.1 Scope	31
404.2 Materials.....	31
404.3 Design Loads.....	31
404.4 Structural System and Load Paths	31
404.5 Structural Analysis	32
404.6 Strength	32
404.7 Serviceability.....	32
404.8 Durability	32
404.9 Sustainability	33
404.10 Structural Integrity	33
404.11 Fire Resistance	33
404.12 Requirements for Specific Types of Construction.....	33
404.13 Construction and Inspection	34
404.14 Strength Evaluation of Existing Structures	34
SECTION 405	34
LOADS	34
405.1 Scope	34
405.2 General	34
405.3 Load Factors and Combinations	34
SECTION 406	36
STRUCTURAL ANALYSIS.....	36

406.1	Scope	36
406.2	General	36
406.3	Modeling Assumptions.....	37
406.4	Arrangement of Live Load	37
406.5	Simplified Method of Analysis for Non-Prestressed Continuous Beams and One-way Slabs	37
406.6	First-order Analysis	38
406.7	Elastic Second-order Analysis	41
406.8	Inelastic Second-Order Analysis	41
406.9	Acceptability of Finite Element Analysis	42
SECTION 407		42
ONE-WAY SLABS		42
407.1	Scope	42
407.2	General	42
407.3	Design Limits	42
407.4	Required Strength	43
407.5	Design Strength	44
407.6	Reinforcement Limits	44
407.7	Reinforcement Detailing.....	45
SECTION 408		47
TWO-WAY SLABS		47
408.1	Scope	47
408.2	General	47
408.3	Design Limits	48
408.4	Required Strength	49
408.5	Design Strength	51
408.6	Reinforcement Limits	52
408.7	Reinforcement Detailing.....	52
408.8	Non-Prestressed Two-Way Joist Systems	56
408.9	Lift-slab Construction	57
408.10	Direct Design Method	57
408.11	Equivalent Frame Method	60
SECTION 409		61
BEAMS		61
409.1	Scope	61
409.2	General	61
409.3	Design Limits	62
409.4	Required Strength	63
409.5	Design Strength	63
409.6	Reinforcement Limits	64
409.7	Reinforcement Detailing.....	65
409.8	Non-Prestressed One-way Joist Systems	68
409.9	Deep Beams	69
SECTION 410		70
COLUMNS		70
410.1	Scope	70
410.2	General	70
410.3	Design Limits	70
410.4	Required Strength	70
410.5	Design Strength	71
410.6	Reinforcement Limits	71

410.7	Reinforcement Detailing	71
SECTION 411		74
WALLS.....		74
411.1	Scope	74
411.2	General	74
411.3	Design Limits	74
411.4	Required Strength.....	75
411.5	Design Strength.....	75
411.6	Reinforcement Limits.....	76
411.7	Reinforcement Detailing	77
411.8	Alternative Method for Out-of-Plane Slender Wall Analysis	78
SECTION 412		79
DIAPHRAGMS		79
412.1	Scope	79
412.2	General	79
412.3	Design Limits	80
412.4	Required Strength.....	80
412.5	Design Strength.....	80
412.6	Reinforcement Limits.....	82
412.7	Reinforcement Detailing	82
SECTION 413		83
FOUNDATIONS.....		83
413.1	Scope	83
413.2	General	83
413.3	Shallow Foundations	84
413.4	Deep Foundations.....	85
SECTION 414		86
PLAIN CONCRETE		86
414.1	Scope	86
414.2	General	86
414.3	Design Limits	86
414.4	Required Strength.....	87
414.5	Design Strength.....	88
414.6	Reinforcement Detailing	89
SECTION 415		90
BEAM-CONCRETE AND.....		90
SLAB-COLUMN JOINTS.....		90
415.1	Scope	90
415.2	General	90
415.3	Transfer of Column Axial Force through the Floor System.....	90
415.4	Detailing of Joints	90
SECTION 416		91
CONNECTION BETWEEN MEMBERS		91
416.1	Scope	91
416.2	Connections of Precast Members	91
416.3	Connections to Foundations	92
416.4	Horizontal Shear Transfer in Composite Concrete Flexural Members	94

416.5	Brackets and Corbels	95
SECTION 417		97
ANCHORING TO CONCRETE		97
417.1	Scope	97
417.2	General	98
417.3	General Requirements for Strength of Anchors.....	100
417.4	Design Requirements for Tensile Loading	102
417.5	Design Requirements for Shear Loading.....	105
417.6	Interaction of Tensile and Shear Forces	108
417.7	Required Edge Distances, Spacings, and Thicknesses to Preclude Splitting Failure.....	108
417.8	Installation and Inspection of Anchors	109
SECTION 418		110
EARTHQUAKE-RESISTANT STRUCTURES		110
418.1	Scope	110
418.4	Intermediate Moment Frames.....	111
418.5	Intermediate Precast Structural Walls.....	113
418.6	Beams of Special Moment Frames	113
418.7	Columns of Special Moment Frames.....	115
418.8	Joints of Special Moment Frames.....	117
418.9	Special Moment Frames Constructed Using Precast Concrete.....	119
418.10	Special Structural Walls	119
418.11	Special Structural Walls Constructed Using Precast Concrete	123
418.12	Diaphragms and Trusses.....	123
418.13	Foundations	125
418.14	Members Not Designated as Part of the Seismic-Force-Resisting System.....	127
SECTION 419		128
CONCRETE: DESIGN AND DURABILITY REQUIREMENTS.....		128
419.1	Scope	128
419.2	Concrete Design Properties	128
419.3	Concrete Durability Requirements	129
419.3.1	Exposure Categories and Classes	129
419.4	Grout Durability Requirements	131
SECTION 420		131
STEEL REINFORCEMENT PROPERTIES, DURABILITY, AND EMBEDMENTS.....		131
420.1	Scope	131
420.2	Non-Prestressed Bars and Wires.....	131
420.3	Prestressing Strands, Wires, and Bars	132
420.5	Headed Shear Stud Reinforcement	136
420.6	Provisions for Durability of Steel Reinforcement	136
420.7	Embedments	138
SECTION 421		139
STRENGTH REDUCTION FACTORS		139
421.1	Scope	139
SECTION 422		142
SECTIONAL STRENGTH.....		142
422.1	Scope	142
422.2	Design Assumptions for Moment and Axial Strength.....	142

422.3	Flexural Strength	143
422.4	Axial Strength or Combined Flexural and Axial Strength	143
422.5	One-way Shear Strength	144
422.6	Two-Way Shear Strength	147
422.7	Torsional Strength	150
422.8	Bearing	152
422.9	Shear Friction	152
SECTION 423		154
STRUT-AND-TIE MODELS		154
423.1	Scope	154
423.2	General	154
423.3	Design Strength	154
423.4	Strength of Struts	154
423.5	Reinforcement Crossing Bottle-Shaped Struts	155
423.6	Strut Reinforcement Detailing	155
423.7	Strength of Ties	155
423.8	Tie Reinforcement Detailing	156
423.9	Strength of Nodal Zones	156
SECTION 424		157
SERVICEABILITY REQUIREMENTS		157
424.1	Scope	157
424.2	Deflections Due to Service-Level Gravity Loads	157
424.3	Distribution of Flexural Reinforcement in One-Way Slabs and Beams	158
424.4	Shrinkage and Temperature Reinforcement	159
424.5	Permissible Stresses in Prestressed Concrete Flexural Members	160
SECTION 425		161
REINFORCEMENT DETAILS		161
425.1	Scope	161
425.2	Minimum Spacing of Reinforcement	161
425.3	Standard Hooks, Seismic Hooks, Crossties, and Minimum Inside Bend Diameters	162
425.4	Development of Reinforcement	163
425.5	Splices	167
425.6	Bundled Reinforcement	169
425.7	Transverse Reinforcement	169
425.8	Post-Tensioning Anchorages and Couplers	172
425.9	Anchorage Zones for Post-Tensioned Tendons	172
SECTION 426		175
CONSTRUCTION DOCUMENTS AND INSPECTION		175
426.1	Scope	175
426.2	Design Criteria	175
426.3	Member Information	175
426.4	Concrete Materials and Mixture Requirements	175
426.5	Concrete Production and Construction	178
426.6	Reinforcement Materials and Construction Requirements	181
426.7	Anchoring to Concrete	182
426.8	Embedments	182
426.9	Additional Requirements for Precast Concrete	183
426.10	Additional Requirements for Prestressed Concrete	183
426.11	Formwork	184
426.12	Concrete Evaluation and Acceptance	185

422.3	Flexural Strength	143
422.4	Axial Strength or Combined Flexural and Axial Strength	143
422.5	One-way Shear Strength	144
422.6	Two-Way Shear Strength	147
422.7	Torsional Strength	150
422.8	Bearing	152
422.9	Shear Friction	152
SECTION 423		154
STRUT-AND-TIE MODELS		154
423.1	Scope	154
423.2	General	154
423.3	Design Strength	154
423.4	Strength of Struts	154
423.5	Reinforcement Crossing Bottle-Shaped Struts	155
423.6	Strut Reinforcement Detailing	155
423.7	Strength of Ties	155
423.8	Tie Reinforcement Detailing	156
423.9	Strength of Nodal Zones	156
SECTION 424		157
SERVICEABILITY REQUIREMENTS		157
424.1	Scope	157
424.2	Deflections Due to Service-Level Gravity Loads	157
424.3	Distribution of Flexural Reinforcement in One-Way Slabs and Beams	158
424.4	Shrinkage and Temperature Reinforcement	159
424.5	Permissible Stresses in Prestressed Concrete Flexural Members	160
SECTION 425		161
REINFORCEMENT DETAILS		161
425.1	Scope	161
425.2	Minimum Spacing of Reinforcement	161
425.3	Standard Hooks, Seismic Hooks, Crossties, and Minimum Inside Bend Diameters	162
425.4	Development of Reinforcement	163
425.5	Splices	167
425.6	Bundled Reinforcement	169
425.7	Transverse Reinforcement	169
425.8	Post-Tensioning Anchorages and Couplers	172
425.9	Anchorage Zones for Post-Tensioned Tendons	172
SECTION 426		175
CONSTRUCTION DOCUMENTS AND INSPECTION		175
426.1	Scope	175
426.2	Design Criteria	175
426.3	Member Information	175
426.4	Concrete Materials and Mixture Requirements	175
426.5	Concrete Production and Construction	178
426.6	Reinforcement Materials and Construction Requirements	181
426.7	Anchoring to Concrete	182
426.8	Embedments	182
426.9	Additional Requirements for Precast Concrete	183
426.10	Additional Requirements for Prestressed Concrete	183
426.11	Formwork	184
426.12	Concrete Evaluation and Acceptance	185

426.13	Inspection	187
SECTION 427		188
STRENGTH EVALUATION OF EXISTING STRUCTURES.....		188
427.1	Scope	188
427.2	General	188
427.3	Analytical Strength Evaluation.....	189
427.4	Strength Evaluation by Load Test	190
427.5	Reduced Load Rating	191
SECTION 428		191
BUILDING CODE REQUIREMENTS FOR CONCRETE THIN SHELLS.....		191
428.1	Scope and Definitions.....	191
428.2	Analysis and Design.....	192
428.3	Design Strength	192
428.4.1	Cast-in-place Non-Prestressed Concrete.....	192
428.4.2	Cast-in-place Prestressed Concrete	192
428.4.3	Precast Non-Prestressed or Prestressed Concrete Manufactured Under Plant Conditions.....	193
428.4.4	Specified Concrete Cover Requirements for Corrosive Environments.....	193
428.5	Shell Reinforcement	194
428.6	Construction	194
SECTION 429		194
ALTERNATE DESIGN METHOD		194
429.1	Notations.....	194
429.2	Scope	194
429.3	General	194
429.4	Permissible Service Load Stresses.....	195
429.5	Development and Splices of Reinforcement.....	195
429.7	Compression Members With or Without Flexure.....	195
429.8	Shear and Torsion.....	199
APPENDIX A.....		201
APPENDIX B		201

SECTION 401

GENERAL REQUIREMENTS

401.1 Scope

401.1.1 This section addresses (a) to (h):

- a. General requirements of this Chapter;
- b. Purpose of this Chapter;
- c. Applicability of this Chapter;
- d. Interpretation of this Chapter;
- e. Definition of building official and the licensed design professional;
- f. Construction documents;
- g. Testing and inspection;
- h. Approval of special systems of design, construction, or alternative construction materials.

401.2 General

401.2.1 Chapter 4 refers to the structural concrete provision of the National Structural Code of the Philippines, Volume I (NSCP Vol. I), 7th Edition and may be cited as such, and will be referred to herein as “this Code”.

401.2.2 This chapter provides minimum requirements for the design and construction of structural concrete elements of any building or other structure under requirements of the National Building Code of the Philippines of which this chapter of the National Structural Code of the Philippines, Volume I, forms a part. This chapter also covers the strength evaluation of existing concrete structures.

For structural concrete, f'_c shall not be less than 17 MPa. No maximum value of f'_c shall apply unless restricted by a specific code provision.

401.2.3 This chapter is in English, with SI units, published by the Association of Structural Engineers of the Philippines, Inc.

401.2.4 In case of conflict between this edition and other earlier versions, this latest version governs.

401.2.5 This chapter provides the minimum requirements for the materials, design, construction, and strength evaluation of structural concrete members and systems in any structure designed and constructed under the requirements of the general building code.

401.2.6 Modifications to this Code that are adopted by a particular government agency or local government are part of that organization's requirements, but are not part of this Code.

401.2.7 This chapter provides the minimum requirements for the materials, design, construction, and strength evaluation of structural concrete members and systems in any structure within this Code.

401.3 Purpose

401.3.1 The purpose of this chapter is to provide for public health and safety by establishing minimum requirements for strength, stability, serviceability, durability, and integrity of concrete structures.

401.3.2 This chapter does not address all design considerations.

401.3.3 Construction means and methods are not addressed in this section.

401.4 Applicability

401.4.1 This chapter shall apply to concrete structures designed and constructed under the requirements of the general building code.

401.4.2 Applicable provisions of this chapter shall be permitted to be used for structures not governed by the general building code.

401.4.3 The design of thin shells and folded plate concrete structures shall be in accordance with Section 428.

401.4.4 Design and construction of structural concrete slabs cast on stay-in-place, non-composite steel deck are governed by this chapter.

401.4.5 Design and construction of one- and two-family dwellings and multiple single-family dwellings (townhouses) and their accessory structures may be designed until such time provisions of the National Structural Code of the Philippines, Volume III, Housing is published.

401.4.6 This chapter does not apply to the design and installation of concrete piles, drilled piers, and caissons embedded in ground except as provided in (a) or (b):

- a. For portions in air or water, or in soil incapable of providing adequate lateral restraint to prevent buckling throughout their length. See also Sect. 418.13.1.2.
- b. For structures in region of high seismic risk or assigned to high seismic performance or design categories

401.4.7 This chapter does not govern design and construction of slabs-on-ground, unless the slab transmits vertical loads or lateral forces from other portions of the structure to the soil.

401.4.8 For unusual structures, such as arches, tanks, reservoirs, bins and silos, blast-resistant structures, and chimneys, provisions of this section shall govern where applicable. For tanks and reservoirs refer also to ACI 350, ACI 334.1R, and ACI 372R.

401.4.9 This chapter does not govern the composite design of structural concrete slabs cast-in-place, composite steel form deck. Concrete used in the construction of such slabs shall be governed by Sections 401 to 406 of this chapter, where applicable. Portions of such slabs designed as reinforced concrete are governed by this chapter.

401.5 Interpretation

401.5.1 The principles of interpretation in this Section shall apply to this Chapter as a whole unless otherwise stated.

401.5.2 This Chapter consists of sections and appendices, including text, headings, tables, figures, footnotes to tables and figures, and referenced standards.

401.5.3 This Chapter shall be interpreted in a manner that avoids conflict between or among its provisions. Specific provisions shall govern over general provisions.

401.5.4 This chapter shall be interpreted and applied in accordance with the plain meaning of the words and terms used. Specific definitions of words and terms in this section shall be used where provided and applicable, regardless of whether other materials, standards, or resources outside of this section provide a different definition.

401.5.5 The following words and terms in this section shall be interpreted in accordance with (a) through (e):

- a. The word “shall” is always mandatory;
- b. Provisions of this section are mandatory even if the word “shall” is not used;
- c. Words used in the present tense shall include the future;
- d. The word “and” indicates that all of the connected items, conditions, requirements, or events shall apply;
- e. The word “or” indicates that all of the connected items, conditions, requirements, or events are alternatives, at least one of which shall be satisfied.

401.5.6 In any case in which one or more provisions of this Chapter are declared by an appropriate court to be invalid, that ruling shall not affect the validity of the remaining provisions of this Chapter, which are severable. The ruling of the court shall be effective only in that court’s jurisdiction, and shall not affect the content of interpretation of this Chapter in other jurisdictions.

401.5.7 If conflicts occur between provisions of this code and standards and documents referenced in Section 403, this Chapter shall apply.

401.6 Building Official

401.6.1 All references in this chapter to the building official shall be understood to mean the persons who administer and enforce this Code.

401.6.2 Actions and decisions by the building official affect only the specific jurisdiction and do not change this Code.

401.6.3 The building official shall have the right to order testing of any materials used in concrete construction to determine if materials are of the quality specified.

401.7 Licensed Design Professional

401.7.1 All references in this Code to the licensed design professional shall be understood to mean the person who is licensed and responsible for, and in charge of, the structural design or inspection.

401.8 Construction Documents and Design Records

401.8.1 The licensed design professional shall provide in the construction documents the information required in Section 426 and that required by the jurisdiction.

401.8.2 Calculations pertinent to design shall be filed with the construction documents if required by the building official. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. Model analysis shall be permitted to supplement calculations.

401.9 Testing and Inspection

401.9.1 Concrete materials shall be tested in accordance with the requirements of Section 426.

401.9.2 Concrete construction shall be inspected in accordance with the general building code and in accordance with Sections 417 and 426.

401.9.3 Inspection records shall include information required in Sections 417 and 426.

401.10 Approval of Special Systems of Design, Construction, or Alternative Construction Materials

401.10.1 Sponsors of any system of design, construction, or alternative construction materials within the scope of this Chapter, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by this Chapter, shall have the right to present the data on which their design is based to the building official or to a committee of competent structural engineers appointed by the building official. This committee shall have the authority to investigate the data so submitted, require tests, and formulate rules governing design and construction of such systems to meet the intent of this Code. These rules, when approved by the building official, and promulgated, shall be of the same force and effect as the provisions of this Code.

401.11 Provisions for Earthquake Resistance

401.11.1 In regions of moderate (seismic zone 2) or high seismic risk (seismic zone 4), provisions of Section 418 shall be satisfied.

SECTION 402 NOTATION AND TERMINOLOGY

402.1 Scope

402.1.1 This section defines notation and terminology used in this chapter.

402.2 Notation

a	= shear span, distance between concentrated load and face of supports, mm.
a	= depth of equivalent rectangular stress block, mm.
a_v	= shear span, equal to distance from center of concentrated load to either: (a) face of support for continuous or cantilevered members, or (b) center of support for simply supported members, mm.
A	= area of that part of cross section between flexural tension face and center of gravity of gross section, mm ² .
A_b	= Area of an individual bar or wire, mm ² .
A_{brg}	= net bearing area of the head of stud, anchor bolt, or headed deformed bars, mm ² .
A_c	= area of concrete section resisting shear transfer, mm ² . = area of contact surface being investigated for horizontal shear, mm ² .
A_{cf}	= greater gross cross-sectional area of the slab-beam strips of the two orthogonal equivalent frames intersecting at a column of a two-way slab, mm ² .
A_{ch}	= cross-sectional area of a member measured to the outside edges of transverse reinforcement, mm ² .
A_{cp}	= area enclosed by outside perimeter of concrete cross section, mm ² .
A_{cs}	= cross-sectional area at one end of a strut in a strut-and-tie model, taken perpendicular to the axis of the strut, mm ² .
A_{ct}	= area of that part of cross section between the flexural tension face and centroid of gross section, mm ² .

A_{cv}	= gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered in the case of walls, and gross area of concrete section in the case of diaphragms, not to exceed the thickness times the width of the diaphragm, mm^2 .	A_{oh}	= area enclosed by centerline of the outermost closed transverse torsional reinforcement, mm^2 .
A_{cw}	= area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear, mm^2 .	A_{pd}	= total area occupied by duct, sheathing, and prestressing reinforcement, mm^2 .
A_f	= area of reinforcement in bracket or corbel resisting design moment, mm^2 .	A_{ps}	= area of prestressed reinforcement in tension zone, mm^2 .
A_g	= gross area of concrete section, mm^2 . For a hollow section, A_g is the area of the concrete only and does not include the area of the void(s).	A_{pt}	= total area of prestressing reinforcement, mm^2 .
A_h	= total area of shear reinforcement parallel to primary tension reinforcement in a corbel or bracket, mm^2 .	A_s	= area of non-prestressed longitudinal tension reinforcement, mm^2 .
A_j	= effective cross-sectional area within a joint in a plane parallel to plane of beam reinforcement generating shear in the joint, mm^2 .	A'_s	= area of compression reinforcement, mm^2 .
A_l	= total area of longitudinal reinforcement to resist torsion, mm^2 .	A_{sc}	= area of primary tension reinforcement in a corbel or bracket, mm^2 .
$A_{l,min}$	= minimum area of longitudinal reinforcement to resist torsion, mm^2 .	$A_{se,N}$	= effective cross-sectional area of anchor intension, mm^2 .
A_n	= area of reinforcement in bracket or corbel resisting factored tensile force N_{uc} , mm^2 .	$A_{se,V}$	= effective cross-sectional area of anchor in shear, mm^2 .
A_{nz}	= area of a face of a nodal zone or a section through a nodal zone, mm^2 .	A_{sh}	= total cross-sectional area of transverse reinforcement, including crossties, within spacing s and perpendicular to dimension b_c , mm^2 .
A_{Na}	= projected influence area of a single adhesive anchor or group of adhesive anchors, for calculation of bond strength intension, mm^2 .	A_{si}	= total area of surface reinforcement at spacing s_i in the i -th layer crossing a strut, with reinforcement at an angle α_i to the axis of the strut, mm^2 .
A_{Nao}	= projected influence area of a single adhesive anchor, for calculation of bond strength in tension if not limited by edge distance or spacing, mm^2 .	$A_{s,min}$	= minimum area of flexural reinforcement, mm^2 .
A_{Nc}	= projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension, mm^2 .	A_{st}	= total area of non-prestressed longitudinal reinforcement bars or steel shapes, and excluding prestressing reinforcement, mm^2 .
A_{Nco}	= projected concrete failure area of a single anchor, for calculation of strength intension if not limited by edge distance or spacing, mm^2 .	A_{sx}	= area of steel shape, pipe, or tubing in a composite section, mm^2 .
A_o	= gross area enclosed by torsional shear flow path, mm^2 .	A_t	= area of structural steel shape, pipe or tubing in a composite section, mm^2 .
		A_t	= total area of longitudinal reinforcement to resist torsion, mm^2 .
		A_{tp}	= area of prestressing reinforcement in a tie, mm^2 .
		A_{tr}	= total cross-sectional area of all transverse reinforcement within spacing s that crosses the potential plane of splitting through the reinforcement being developed, mm^2 .
		A_{ts}	= area of non-prestressed reinforcement in a tie, mm^2 .
		A_v	= area of shear reinforcement within spacing s , mm^2 .
		A_{vd}	= total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam,

A_{vf}	= area of shear friction reinforcement, mm^2 .		
A_{vh}	= area of shear reinforcement parallel to flexural tension reinforcement within spacing s_2 , mm^2 .	B_n	= nominal bearing strength, N.
$A_{v,min}$	= minimum area of shear reinforcement within spacing s , mm^2 .	B_u	= factored bearing load, N.
A_{Vc}	= projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear, mm^2 .	c	= distance from extreme compression fiber to neutral axis, mm.
A_{Vco}	= projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness, mm^2 .	c_{ac}	= critical edge distance required to develop the basic strength as controlled by concrete breakout or bond of a post-installed anchor in tension in uncracked concrete without supplementary reinforcement to control splitting, mm.
A_1	= loaded area for consideration of bearing strength, mm^2 .	$c_{a,max}$	= maximum distance from center of an anchor shaft to the edge of concrete, mm.
A_2	= maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area.	$c_{a,min}$	= minimum distance from center of an anchor shaft to the edge of concrete, mm.
A_2	= the area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having its upper base equal to the loaded area. The sides of the pyramid, cone, or tapered wedge shall be sloped one vertical to two horizontal, mm^2 .	c_{a1}	= distance from the center of an anchor shaft to the edge of concrete in one direction, mm. If shear is applied to anchor, c_{a1} is taken in the direction of the applied shear. If tension is applied to the anchor, c_{a1} is the minimum edge distance. Where anchors subject to shear are located in narrow sections of limited thickness, See Section 417.5.2.4.
b	= width of compression face of member, mm.	c'_{a1}	= limiting value of c_{a1} where anchors are located less than $1.5c_{a1}$ from three or more edges, mm.
b_c	= cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh} , mm.	c_{a2}	= distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to c_{a1} , mm.
b_f	= Effective flange width of T section, mm.	c_b	= lesser of: (a) the distance from center of a bar or wire to the nearest concrete surface, and (b) one-half the center-to-center spacing of bars or wires being developed, mm.
b_o	= perimeter of critical section for two-way shear in slabs and footings, mm.	c_c	= clear cover of reinforcement, mm.
b_s	= width of a strut, mm.	c_{Na}	= projected distance from center of an anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor, mm.
b_{slab}	= the effective slab width resisting $\gamma_f M_{sc}$.	c_t	= distance from the interior face of the column to the slab edge measured parallel to c_1 , but not exceeding c_1 , mm.
b_t	= width of that part of cross section containing the closed stirrups resisting torsion, mm.	c_1	= dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm.
b_v	= width of cross section at contact surface being investigated for horizontal shear, mm.	c_2	= dimension of rectangular or
b_w	= web width or diameter of circular section, mm.		
b_1	= dimension of the critical section b_o measured in the direction of the span for which moments are determined, mm.		
b_2	= dimension of the critical section b_o		

	equivalent rectangular column, capital, or bracket measured in the direction perpendicular to c_1 , mm.				concrete, MPa.
C	= cross-sectional constant to define torsional properties of slab and beam.	EI	= flexural stiffness of member, N-mm ² .		
	= compressive force acting on a nodal zone, N.	$(EI)_{eff}$	= Effective flexural stiffness of member, N-mm ² .		
C_m	= a factor relating actual moment diagram to an equivalent uniform moment diagram.	E_p	= modulus of elasticity of prestressing reinforcement, MPa.		
d	= distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm.	E_s	= modulus of elasticity of reinforcement and structural steel, excluding prestressing reinforcement, MPa.		
d'	= distance from extreme compression fiber to centroid of longitudinal compression reinforcement, mm.	f'_c	= specified compressive strength of concrete, MPa.		
d_a	= outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, mm.	$\sqrt{f'_c}$	= square root of specified compressive strength of concrete, MPa.		
d'_a	= value substituted for d_a if an oversized anchor is used, mm.	f_{ce}	= effective compressive strength of the concrete in a strut or a nodal zone, MPa.		
d_{agg}	= Nominal maximum size of coarse aggregate, mm.	$f_{ci'}$	= compressive strength of concrete at time of initial prestress, MPa.		
d_b	= nominal diameter of bar, wire, or prestressing strand, mm.	$\sqrt{f'_{ci}}$	= square root of specified compressive strength of concrete at time of initial prestress, MPa.		
d_p	= distance from extreme compression fiber to centroid of prestressed reinforcement, mm.	f_{cm}	= measured average compressive strength of concrete, MPa.		
d_{pile}	= diameter of pile at footing base, mm.	f_{ct}	= average splitting tensile strength of lightweight concrete, MPa.		
d_s	= distance from extreme tension fiber to centroid of tension reinforcement, mm.	f_d	= stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, MPa.		
D	= dead loads or related internal moments and forces.	f_{dc}	= decompression stress; stress in the prestressing steel when stress is zero in the concrete at the same level as the centroid of the prestressing steel, MPa.		
D	= effect of service dead load.	f_{pc}	= compressive stress in concrete, after allowance for all prestress losses, at centroid of cross section resisting externally applied loads or at junction of web and flange where the centroid lies within the flange, MPa. In a composite member, f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange where the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone.		
e_h	= distance from the inner surface of the shaft of a J- or L-bolt to the outer tip of the J- or L-bolt, mm.	f_{pe}	= compressive stress in concrete due to effective prestress forces, after allowance for all prestress losses, at extreme fiber of section if tensile stress is caused by externally applied loads, MPa.		
$e_{N'}$	= distance between resultant tension load on a group of anchors loaded in tension and the centroid of the group of anchors loaded in tension, mm; $e_{N'}$ is always positive.	f_{ps}	= stress in prestressing steel at nominal flexural strength, MPa.		
$e_{V'}$	= distance between resultant shear load on a group of anchors loaded in shear in the same direction, and the centroid of the group of anchors loaded in shear in the same direction, mm; $e_{V'}$ is always positive.				
E	= effect of horizontal and vertical earthquake-induced forces.				
E_c	= modulus of elasticity of concrete, MPa.				
E_{cb}	= modulus of elasticity of beam concrete, MPa.				
E_{cs}	= modulus of elasticity of slab				

f_{pu}	= specified tensile strength of prestressing reinforcement, MPa.	h_{sx}	= storey height for storey x .
f_{py}	= specified yield strength of prestressing reinforcement, MPa.	h_u	= laterally unsupported height at extreme compression of wall or wall pier, mm., equivalent to ℓ_u for compression members, mm.
f_r	= modulus of rupture of concrete, MPa.	h_v	= depth of shear head cross section, mm.
f_s	= tensile stress in reinforcement at service loads, excluding prestressing reinforcement, MPa.	h_w	= height of entire wall from base to top, or clear height of wall segment or wall pier considered, mm.
f'_s	= compressive stress in reinforcement under factored loads, excluding prestressing reinforcement, MPa.	h_x	= maximum center to center horizontal spacing of hoop or crosstie legs on all faces of the column, mm.
f_{se}	= effective stress in prestressed reinforcement (after allowance for all prestress losses), MPa.	H	= effect of service load due to lateral earth pressure, ground water pressure, or pressure of bulk materials, N.
f_{si}	= stress in the i -th layer of surface reinforcement, MPa.	I	= moment of inertia of section of beam about the centroidal axis, mm ⁴ .
f_t	= extreme fiber stress in tension in the pre-compressed tensile zone calculated at service loads using gross section properties after allowance of all prestress losses, MPa.	I_b	= moment of inertia about centroidal axis of gross section of beam, mm ⁴ .
f_{uta}	= specified tensile strength of anchor steel, MPa.	I_{cr}	= moment of inertia of cracked section transformed to concrete, mm ⁴ .
f_y	= specified yield strength of non-prestressed reinforcement, MPa.	I_e	= effective moment of inertia for computation of deflection, mm ⁴ .
f_{ya}	= specified yield strength of anchor steel, MPa.	I_g	= moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm ⁴ .
f_{yt}	= specified yield strength of transverse reinforcement, MPa.	I_s	= moment of inertia about of slab about centroidal axis, mm ⁴ .
F	= effect of service lateral load due to fluids with well-defined pressures and maximum heights.	I_{se}	= moment of inertia of reinforcement about centroidal axis of member cross section, mm ⁴ .
F_{nn}	= nominal strength at face of a nodal zone, N.	I_{sx}	= moment of inertia of structural steel shape, pipe or tubing about centroidal axis of composite member cross section, mm ⁴ .
F_{ns}	= nominal strength of a strut, N.	k	= effective length factor for compression members.
F_{nt}	= nominal strength of a tie, N.	k_c	= coefficient for basic concrete breakout strength in tension.
F_{un}	= factored force on the face of a node, N.	k_{cp}	= coefficient for pry out strength.
F_{us}	= factored compressive force in a strut, N.	k_f	= concrete strength factor.
F_{ut}	= factored tensile force in a tie, N.	k_n	= confinement effectiveness factor.
h	= overall thickness of member, mm.	K	= wobble friction coefficient per mm of prestressing tendon.
h	= thickness of shell or folded plate, mm.	K_t	= torsional stiffness of torsional member; moment per unit rotation.
h_a	= thickness of member in which an anchor is located, measured parallel to anchor axis, mm.	K_{tr}	= transverse reinforcement index, mm.
h_{anc}	= dimension of anchorage device or single group of closely spaced devices in the direction of bursting being considered, mm.	K_{05}	= coefficient associated with the 5% fractile.
h_{ef}	= effective embedment depth of anchor, mm.	ℓ	= span length of beam or one-way slab; clear projection of cantilever, mm.
h'_{ef}	= limiting value of h_{ef} where anchors are located less than $1.5h_{ef}$ from three or more edges, mm.	ℓ_a	= additional embedment length beyond centerline of support or point of

	inflection, mm.	ℓ_2	= length of span in direction perpendicular to ℓ_1 , measured center-to-center of supports, mm.
ℓ_{anc}	= length along which anchorage of a tie must occur, mm.	L	= effect of service live load.
ℓ_b	= width of bearing, mm.	L_d	= development length, mm.
ℓ_c	= length of a compression member in a frame, measured center-to-center of the joints, mm.	L_r	= effect of service roof live load.
ℓ_d	= development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, or pretensioned strand, mm.	M	= design moment
ℓ_{dc}	= development length in compression of deformed bars and deformed wire, mm.		= maximum unfactored moment due to service loads, including P_{Δ} effects.
ℓ_{dh}	= development length in tension of deformed bar or deformed wire with a standard hook, measured from outside end of hook, point of tangency, toward critical section, mm.	M	= moment acting on anchor or anchor group, N-mm.
ℓ_e	= load bearing length of anchor for shear, mm.	M_a	= maximum moment in member due to service loads at stage deflection is calculated, N-mm.
ℓ_n	= length of clear span measured face-to-face of supports, mm.	M_c	= moment at the face of the joint, corresponding to the nominal flexural strength of the column framing into that joint, calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.
ℓ_o	= length, measured from joint face along axis of member, over which special transverse reinforcement must be provided, mm.	M_c	= factored moment amplified for the effects of member curvature used for design of compression member, N-mm.
ℓ_{sc}	= compression lap splice length, mm.	M_{cr}	= cracking moment, N-mm.
ℓ_{st}	= tension lap splice length, mm.	M_{cre}	= moment causing flexural cracking at section due to externally applied loads, N-mm
ℓ_t	= span of member under load test, taken as the shorter span for two-way slab systems, mm. Span is the lesser of: (a) distance between centers of supports, and (b) clear distance between supports plus thickness h of member. Span for a cantilever shall be taken as twice the distance from the face of support to cantilever end.	M_g	= moment at the face of the joint, corresponding to the nominal flexural strength of the girder including slab where in tension, framing into that joint
ℓ_{tr}	= transfer length of prestressed reinforcement, mm.	M_m	= factored moment modified to account for effect of axial compression, N-mm.
ℓ_u	= unsupported length of column or wall, mm	M_{max}	= maximum factored moment at section due to externally applied loads, N-mm.
ℓ_v	= length of shear head arm from centroid of concentrated load or reaction, mm.	M_n	= nominal flexural strength at section, N-mm.
ℓ_w	= length of entire wall, or length of wall segment or wall pier considered in direction of shear force, mm.	M_{nb}	= nominal flexural strength of beam including slab where in tension, framing into joint, N-mm.
ℓ_x	= length of prestressing tendon element from jacking end to any point x , mm.	M_{nc}	= nominal flexural strength of column framing into joint, calculated for factored axial force, consistent with the direction of lateral forces considered, resulting in lowest flexural strength, N-mm.
ℓ_1	= length of span in direction that moments are being determined, measured center-to-center of supports, mm.	M_o	= total factored static moment, N-mm.
		M_p	= required plastic moment strength of shearhead cross section, N-mm.
		M_{pr}	= probable flexural strength of

	members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least $1.25f_y$ and a strength-reduction factor ϕ of 1.0, N-mm.	n	= modular ratio of elasticity, but not less than 6 = E_s/E_c
M_{sa}	= maximum moment in wall due to service loads, excluding P_Δ effects, N-mm.	n	= number of items, such as, bars, wires, monostrand anchorage devices, anchors or shearhead arms.
M_{sc}	= factored slab moment that is resisted by the column at a joint, N-mm.	n_l	= number of longitudinal bars around the perimeter of a column core with rectilinear hoops that are laterally supported by the corner of hoops or by seismic hooks. A bundle of bars is counted as a single bar.
M_{slab}	= portion of slab factored moment balanced by support moment, N-mm	N	= design axial load normal to cross section occurring simultaneously with V ; to be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage.
M_u	= factored moment at section, N-mm.		= number of bars in a layer being spliced or developed at a critical section.
M_{ua}	= moment at the mid-height section of the wall due to factored lateral and eccentric vertical loads, not including P_Δ effects, N-mm.	N	= tension force acting on anchor or anchor group, N.
M_v	= moment resistance contributed by shearhead reinforcement, N-mm.	N_a	= nominal bond strength intension of a single adhesive anchor, N.
M_1	= lesser factored end moment on a compression member, to be taken as positive if member is bent in single curvature, negative if bent in double curvature, N-mm.	N_{ag}	= nominal bond strength intension of a group of adhesive anchors, N.
M_{1ns}	= factored end moment on a compression member at the end at which M_1 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm.	N_b	= basic concrete breakout strength in tension of a single anchor in cracked concrete, N.
M_{1s}	= factored end moment on compression member at the end at which M_1 acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm.	N_{ba}	= Basic bond strength in tension of a single adhesive anchor, N.
M_2	= greater factored end moment on compression member. If transverse loading occurs between supports, M_2 is taken as the largest moment occurring in member. Value of M_2 is always positive, N-mm.	N_c	= the resultant tensile force acting on the portion of the concrete cross section that is subjected to tensile stresses due to the combined effects of service loads and effective prestress, N.
$M_{2,min}$	= minimum value of M_2 , N-mm.	N_{cb}	= nominal concrete breakout strength in tension of a single anchor, N.
M_{2ns}	= factored end moment on compression member at the end at which M_2 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm.	N_{cbg}	= nominal concrete breakout strength in tension of a group of anchors, N.
M_{2s}	= factored end moment on compression member at the end at which M_2 acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm.	N_{cp}	= basic concrete pryout strength of a single anchor, N.
		N_{cpg}	= basic concrete pryout strength of a group of anchors, N.
		N_n	= nominal strength in tension, N.
		N_p	= pullout strength in tension of a single anchor in cracked concrete, N.
		N_{pn}	= nominal pullout strength in tension of a single anchor, N
		N_{sa}	= nominal strength of a single anchor or individual anchors in a group of anchors in tension as governed by the steel strength, N.

N_{sb}	= side-face blowout strength of a single anchor, N.	$P\delta$	= secondary moment due to individual member slenderness, N-mm.
$N_{sb,g}$	= side-face blowout strength of a group of anchors, N.	$P\Delta$	= secondary moment due to lateral deflection, N-mm.
N_u	= factored axial force normal to cross section occurring simultaneously with V_u or T_u ; to be taken as positive for compression and negative for tension, N.	P_x	= prestressing tendon force at any point x .
N_{ua}	= factored tensile force applied to anchor or individual anchor in a group of anchors, N.	q_{Du}	= factored dead load per unit area, kPa.
$N_{ua,g}$	= total factored tensile force applied to anchor group, N.	q_{Lu}	= factored live load per unit area, kPa.
$N_{ua,i}$	= factored tensile force applied to most highly stressed anchor in a group of anchors, N.	q_u	= factored load per unit area, kPa.
$N_{ua,s}$	= Factored sustained tension load, N.	Q	= stability index for a storey.
N_{uc}	= factored horizontal tensile force applied at top of bracket or corbel acting simultaneously with V_u ; to be taken as positive for tension, N.	r	= radius of gyration of cross section of a compression member, mm.
p_{cp}	= outside perimeter of the concrete cross section, mm.	R	= reaction, N.
p_h	= perimeter of centerline of outermost closed transverse torsional reinforcement, mm.	R	= cumulative load effect of service rain load.
P_c	= critical buckling load, N.	s	= spacing of shear reinforcement in direction parallel to longitudinal reinforcement, mm.
P_n	= nominal axial compressive strength of a member, N.	s	= center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, tendons, or anchors, mm.
$P_{n,max}$	= maximum nominal axial compressive strength of member, N.	s	= Standard deviation, MPa.
P_{nt}	= nominal axial tensile strength of member, N.	s_i	= center-to-center spacing of reinforcements in the i -th layer adjacent to the surface of the member, mm.
$P_{nt,max}$	= Maximum nominal axial tensile strength of member, N.	s_o	= center-to-center spacing of transverse reinforcements within the length ℓ_o , mm.
P_o	= nominal axial load strength at zero eccentricity, N.	s_s	= sample standard deviation, MPa.
P_{pj}	= prestressing force at jacking end, N.	s_w	= spacing of wire to be developed or spliced, mm.
P_{pu}	= factored prestressing force at anchorage device, N.	s_w	= clear distance between adjacent webs, mm.
P_{px}	= prestressing force evaluated at distance ℓ_{px} from the jacking end, N.	s_2	= center-to-center spacing of longitudinal shear or torsional reinforcement, mm.
P_s	= prestressing tendon force at jacking end.	S	= elastic section modulus of section.
P_s	= unfactored axial load at the design, mid-height section including effects of self-weight, N.	S_e	= moment, shear or axial force at connection corresponding to development of probable strength at intended yield locations, based on the governing mechanism of inelastic lateral deformation, considering both gravity and earthquake load effects.
P_{su}	= factored post-tensioned tendon force at the anchorage device.	S_m	= elastic section modulus, mm^3 .
P_u	= factored axial load at given eccentricity, $\leq \phi P_n$.	S_n	= nominal moment, shear, axial, torsional, or bearing strength.
P_u	= Factored axial force; to be taken as positive for compression and negative for tension, N.	S_y	= yield strength of connection, based on f_y , for moment, shear, or axial force, MPa.
		t	= wall thickness of hollow section, mm.
		t_f	= thickness of flange, mm.

T	= cumulative effects of service temperature, creep, shrinkage, differential settlement, and shrinkage compensating concrete.	V_{cpg}	= nominal concrete pryout strength of a group of anchors, N.
T	= tension force acting on a nodal zone in a strut-and-tie model, N.	V_{cw}	= nominal shear strength provided by concrete where diagonal cracking results from high principal tensile stress in web, N.
T_{cr}	= cracking torsional moment, N-mm.	V_d	= shear force at section due to unfactored dead load, N.
T_t	= total test load, N.	V_e	= Design shear force for load combinations including earthquake effects, N.
T_{th}	= threshold torsional moment, N-mm.	V_h	= permissible horizontal shear stress, MPa.
T_u	= factored torsional moment at section, N-mm.	V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max} , N.
U	= strength of a member or cross section required to resist factored loads or related internal moments and forces in such combinations as stipulated in this Code.	V_n	= nominal shear strength, N.
v	= design shear stress, MPa.	V_{nh}	= nominal horizontal shear strength, N.
v_c	= Stress corresponding to nominal two-way shear strength provided by concrete, MPa.	V_p	= vertical component of effective prestress force at section, N.
v_n	= equivalent concrete stress corresponding to nominal two-way shear strength of slab or footing, MPa.	V_s	= nominal shear strength provided by shear reinforcement, N.
v_s	= equivalent concrete stress corresponding to nominal two-way shear strength provided by reinforcement, MPa.	V_{sa}	= nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, N.
v_u	= Maximum factored two-way shear stress calculated around the perimeter of a given critical section, MPa.	V_u	= factored shear force at section, N.
v_{ug}	= factored shear stress on the slab critical section for two-way action due to gravity loads without moment transfer, MPa.	V_{ua}	= factored shear force applied to a single anchor or group of anchors, N.
V	= design shear force at section	$V_{ua,g}$	= total factored shear force applied to anchor group, N.
	= shear force acting on anchor or anchor group, N.	$V_{ua,i}$	= factored shear force applied to most highly stressed anchor in a group of anchors, N.
$V_{\perp}$	= applied shear perpendicular to the edge, N.	V_{uh}	= Factored shear force along horizontal interface in composite concrete flexural member, N.
$V_{\parallel}$	= applied shear parallel to the edge, N.	V_{us}	= factored horizontal shear in a storey, N.
V_b	= basic concrete breakout strength in shear of a single anchor in cracked concrete, N.	w_c	= density, unit weight, of normal-weight concrete or equilibrium density of lightweight concrete, kg/m ³ .
V_c	= nominal shear strength provided by concrete, N.	w_l	= factored live load per unit area.
V_{cb}	= nominal concrete breakout strength in shear of a single anchor, N.	w_u	= factored load per unit length of beam or one-way slab, N/mm.
V_{cbg}	= nominal concrete breakout strength in shear of a group of anchors, N.	w_s	= width of a strut perpendicular to the axis of the strut, mm.
V_{ci}	= nominal shear strength provided by concrete where diagonal cracking results from combined shear and moment, N.	w_t	= effective height of concrete concentric with a tie, used to dimension nodal zone, mm.
V_{cp}	= nominal concrete pryout strength of a single anchor, N.	$w_{t,max}$	= maximum effective height of concrete concentric with a tie, mm.
		w/cm	= maximum water-cementitious materials ratio.
		W	= effect of wind load.
		W_a	= service-level wind load, N.

x	= shorter overall dimension of rectangular part of cross section, mm.	β_{ds}	= the ratio of maximum factored sustained shear within a storey to the maximum factored shear in that storey associated with the same load combination.
y	= longer overall dimension of rectangular part of cross section, mm.	β_{dns}	= ratio used to account for reduction of stiffness of columns due to sustained axial loads.
y_t	= distance from centroidal axis of gross section, neglecting reinforcement, to tension face, mm.	β_n	= factor used to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone.
α	= angle between inclined stirrups and longitudinal axis of member.	β_s	= factor used to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete in a strut.
α	= angle defining the orientation of reinforcement.	β_t	= ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of beam, center-to-center of supports.
α	= total angular change of prestressing tendon profile in radians from tendon jacking end to any point x .	$\beta_t = \frac{E_{cb}C}{2E_{cs}I_s}$	
α_c	= coefficient defining the relative contribution of concrete strength to nominal wall shear strength	β_1	= factor relating depth of equivalent rectangular compressive stress block to depth of neutral axis.
α_f	= ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels, if any, on each side of the beam.	γ_f	= factor used to determine the fraction of M_{sc} transferred by slab flexure at slab-column connections.
$\alpha_f = \frac{E_{cb}I_b}{E_{cs}I_s}$		γ_p	= factor for type of prestressing reinforcement.
α_{fm}	= average value of α_f for all beams on edges of a panel.	γ_p	= 0.55 for f_{py}/f_{pu} not less than 0.80. = 0.40 for f_{py}/f_{pu} not less than 0.85. = 0.28 for f_{py}/f_{pu} not less than 0.90.
α_{f1}	= α_f in direction of ℓ_1 .	γ_s	= factor used to determine the portion of reinforcement located in center band of footing.
α_{f2}	= α_f in direction of ℓ_2 .	γ_v	= factor used to determine the fraction of M_{sc} transferred by eccentricity of shear at slab-column connections.
α_i	= angle between the axis of a strut and the bars in the i -th layer of reinforcement crossing that strut.	$\gamma_v = 1 - \gamma_f$	
α_{px}	= total angular change of tendon profile from tendon jacking end to point under considerations, radians.	δ	= Moment magnification factor used to reflect effects of member curvature between ends of compression member.
α_s	= constant used to compute V_c in slabs and footings.	δ_s	= moment magnification factor for frames not braced against sideways, to reflect lateral drift resulting from lateral and gravity loads.
α_v	= ratio of flexural stiffness of shear head arm to surrounding composite slab section.	δ_u	= design displacement, mm.
α_1	= orientation of distributed reinforcement in a strut.	Δ_{cr}	= computed, out-of-plane deflection at mid-height of wall corresponding to cracking moment, M_{cr} , mm.
α_2	= orientation of reinforcement orthogonal to α_1 in a strut.	Δ_{fmax}	= maximum deflection measured during the second test relative to the
β	= ratio of long to short dimensions: clear span for two-way slabs, sides of column, concentrated load or reaction area; or sides of a footing.		
β_b	= ratio of area of reinforcement cut off to total area of tension reinforcement at section.		
β_c	= ratio of long side to short side of concentrated load or reaction area		

	position of the structure at the beginning of the second test, mm.	ϵ_{ty}	= value of net tensile strain in the extreme layer of longitudinal tension reinforcement used to define a compression-controlled section.
Δ_{fp}	= increase in stress in prestressing reinforcement due to factored loads, MPa.	θ	= angle between axis of strut, compression diagonal, or compression field and the tension chord of the members.
Δ_{fps}	= stress in prestressing reinforcement at service loads less decompression stress, MPa.	λ	= correction factor related to unit weight of concrete.
Δ_{fpt}	= difference between the stress that can be developed in the strand at the section under consideration and the stress required to resist factored bending moment at section, M_u/ϕ , MPa.	λ	= modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal-weight concrete of the same compressive strength.
Δ_{max}	= measured maximum deflection, mm.	λ_a	= modification factor to reflect the reduced mechanical properties of lightweight concrete in certain concrete anchorage applications.
Δ_n	= calculated, out-of-plane deflection at mid-height of wall corresponding to nominal flexural strength, M_n , mm.	λ_Δ	= multiplier used for additional deflection due to long-term effects.
Δ_o	= relative lateral deflection between the top and bottom of a storey due to V_{us} , mm.	μ	= coefficient of friction.
Δ_r	= residual deflection measured 24 hours after removal of the test load. For the first load test, residual deflection is measured relative to the position of the structure at the beginning of the first load test. For the second load test, residual deflection is measured relative to the position of the structure at the beginning of the second load test, mm.	η	= number of identical arms of shearhead.
Δ_{rmax}	= measured residual deflection, mm.	μ_p	= post-tensioning curvature friction coefficient.
Δ_s	= maximum deflection at or near midheight due to service loads, mm	ξ	= time-dependent factor for sustained load.
Δ_s	= out-of-plane deflection due to service loads, mm.	ρ	= ratio of non-prestressed tension reinforcement.
Δ_u	= calculated out-of-plane deflection at mid-height of wall due to factored loads, mm.	ρ	= $A_s/(l_w d)$
Δ_x	= design storey drift of storey x , mm.	ρ	= ratio of A_s to bd
Δ_1	= maximum deflection, during first load test, measured 24 hours after application of the full test load, mm.	ρ'	= ratio of compression reinforcement, A'_s to bd .
Δ_2	= maximum deflection, during second load test, measured 24 hours after application of the full test load. Deflection is measured relative to the position of the structure at the beginning of second load test, mm.	ρ_b	= reinforcement ratio producing balanced strain conditions.
ϵ_{cu}	= maximum usable strain at extreme concrete compression fiber.	ρ_l	= ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement.
ϵ_t	= net tensile strain in extreme layer of longitudinal tension reinforcement at nominal strength, excluding strains due to effective prestress, creep, shrinkage, and temperature.	ρ_p	= ratio of prestressed reinforcement, A_{ps} to bd_p .
		ρ_s	= ratio of volume of spiral reinforcement to total volume of core confined by the spiral, measured out-to-out of spirals.
		ρ_t	= ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement.
		ρ_v	= ratio of tie reinforcement area to area of contact surface.
		ρ_v	= $\frac{A_v}{b_v s}$
		ρ_w	= ratio of tension reinforcement, A_s to $b_w d$.
		ϕ	= strength-reduction factor.

ϕ_k	= stiffness reduction factor.
τ_{cr}	= characteristic bond stress of adhesive anchor in cracked concrete, MPa.
τ_{uncr}	= characteristic bond stress of adhesive anchor in uncracked concrete, MPa.
ψ_c	= factor used to modify development length based on cover.
$\psi_{c,N}$	= factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete.
$\psi_{cp,N}$	= factor used to modify tensile strength of post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation.
$\psi_{cp,Na}$	= factor used to modify tensile strength of adhesive anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation.
$\psi_{c,P}$	= factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete.
$\psi_{c,V}$	= factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement.
ψ_e	= factor used to modify development length based on reinforcement coating.
$\psi_{ec,N}$	= factor used to modify tensile strength of anchors based on eccentricity of applied loads.
$\psi_{ec,Na}$	= factor used to modify tensile strength of adhesive anchors based on eccentricity of applied loads.
$\psi_{ec,V}$	= factor used to modify shear strength of anchors based on eccentricity of applied loads.
$\psi_{ed,N}$	= factor used to modify tensile strength of anchors based on proximity to edges of concrete member.
$\psi_{ed,Na}$	= factor used to modify tensile strength of adhesive anchors based on proximity to edges of concrete member.
$\psi_{ed,V}$	= factor used to modify shear strength of anchors based on proximity to edges of concrete member.
$\psi_{h,V}$	= factor used to modify shear strength of anchors located in concrete members with $h_a < 1.5c_{a1}$.
ψ_r	= factor used to modify development

length based on confining reinforcement.

ψ_s = factor used to modify development length based on reinforcement size.

ψ_t = factor used to modify development length for casting location intension.

ψ_w = factor used to modify development length for welded deformed wire reinforcement in tension.

Ω_o = amplification factor to account for overstrength of the seismic-force-resisting system determined in accordance with the general building code.

$\omega_w, \omega_{pw}, \omega'_w$ = reinforcement indices for flanged sections computed as for ω , ω_p and ω' except that b shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only.

402.3 Terminology

ADHESIVE are chemical components formulated from organic polymers, or a combination of organic polymers and inorganic materials that cure if blended together.

ADMIXTURE is a material other than water, aggregate, or hydraulic cement used as an ingredient of concrete and added to concrete before or during its mixing to modify its properties.

AGGREGATE is a granular material, such as sand, gravel, crushed stone and iron blast-furnace slag, and when used with a cementing medium forms a hydraulic cement concrete or mortar.

AGGREGATE, LIGHTWEIGHT is an aggregate meeting the requirements of ASTM C330 and having a bulk density with a dry, loose weight of 1120 kg/m^3 or less, determined in accordance with ASTM C29. In some standards, the term lightweight aggregate is being replaced by the term low-density aggregate.

ANCHOR is a steel element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads to the concrete.

ANCHOR, CAST-IN is a headed bolt, headed stud, or hooked bolt (J- or L-bolt) installed before placing concrete.

ANCHOR, HEADED BOLT is a cast-in steel anchor that develops its tensile strength from the mechanical interlock provided by either a head or nut at the embedded end of the anchor.

ANCHOR, HOOKED BOLT is a cast-in anchor anchored mainly by bearing of the 90-degree bend (L-bolt) or 180-degree bend (J-bolt) against the concrete, at its embedded end, and having a minimum e_h equal to $3d_a$.

ANCHOR, HEADED STUD is a steel anchor conforming to the requirements of AWS D1.1M and affixed to a plate or similar steel attachment by the stud arc welding process before casting.

ANCHOR, HORIZONTAL OR UPWARDLY INCLINED is an anchor installed in a hole drilled horizontally or in a hole drilled at any orientation above horizontal.

ANCHOR, POST-INSTALLED, is an anchor installed in hardened concrete; adhesive, expansion, and undercut, anchors are examples of post-installed anchors.

ANCHOR, ADHESIVE is a post-installed anchor, inserted into hardened concrete with an anchor hole diameter not greater than 1.5 times the anchor diameter, that transfers loads to the concrete by bond between the anchor and the adhesive, and bond between the adhesive and the concrete.

ANCHOR, ADHESIVE-STEEL ELEMENTS are steel elements for adhesive anchors include threaded rods, deformed reinforcing bars, or internally threaded steel sleeves with external deformations.

ANCHOR, EXPANSION is a post-installed anchor, inserted into hardened concrete that transfers loads to or from the concrete by direct bearing or friction or both.

ANCHOR, UNDERCUT is a post-installed anchor that develops its tensile strength from the mechanical interlock provided by undercutting of the concrete at the embedded end of the anchor. Undercutting is achieved with a special drill before installing the anchor or alternatively by the anchor itself during its installation.

ANCHOR GROUP is a number of similar anchors having approximately equal effective embedment depths with spacing s between adjacent anchors such that the protected areas overlap.

ANCHOR PULLOUT STRENGTH is the strength corresponding to the anchoring device or a major component of the device sliding out from the concrete without breaking out a substantial portion of the surrounding concrete.

ANCHORAGE DEVICE in post-tensioned members, the hardware used to transfer force from prestressed reinforcement to the concrete.

ANCHORAGE DEVICE, BASIC MONOSTRAND is an anchorage device used with any single strand or a single 16mm. or smaller diameter bar that is in accordance with Sections 425.8.1, 425.8.2 and 425.9.3.1a.

ANCHORAGE DEVICE, BASIC MULTISTRAND is an anchorage device used with multiple strands, bars, or wires, or with single bars larger than 16 mm diameters that satisfies Sections 425.8.1, 425.8.2 and 425.9.3.1b.

ANCHORAGE DEVICE, SPECIAL is an anchorage device that satisfies tests required in Section 425.9.3.1c.

ANCHORAGE ZONE in post-tensioned members, portion of the member through which the concentrated prestressing force is transferred to the concrete and distributed more uniformly across the section; its extent is equal to the largest dimension of the cross section; for anchorage devices located away from the end of a member, the anchorage zone includes the disturbed regions ahead of and behind the anchorage device.

ATTACHMENT is a structural assembly, external to the surface of the concrete that transmits loads to or receives loads from the anchor.

B-REGION is a portion of a member in which it is reasonable to assume that strains due to flexure vary linearly through section.

BASE OF STRUCTURE is a level at which the horizontal earthquake ground motions are assumed to be imparted to a building. This level does not necessarily coincide with the ground level.

BEAM is a member subjected primarily to flexure and shear, with or without axial force or torsion; beams in a moment frame that forms part of the lateral-force-resisting system are predominantly horizontal members; a girder is a beam.

BOUNDARY ELEMENT is a portion along wall and diaphragm edge, including edges of openings, strengthened by longitudinal and transverse reinforcement.

BUILDING OFFICIAL is a term used in a general building code to identify the person charged with administration and enforcement of provisions of the building code. Such term as building inspector is a variation of the title, and the term "building official" as

used in this Code, is intended to include those variations, as well as others that are used in the same sense.

CEMENTITIOUS MATERIALS are materials that have cementing value if used in concrete either by themselves, such as portland cement, blended hydraulic cements, and expansive cement, or such materials in combination with fly ash, raw or other calcined natural pozzolans, silica fume, and slag cement.

COLLECTOR is an element that acts in axial tension or compression to transmit forces between a structural diaphragm and a vertical element of the seismic-force-resisting system.

COLUMN is a member, usually vertical or predominantly vertical, used primarily to support axial compressive load, but that can also resist moment, shear, or torsion. Columns used as part of a lateral-force-resisting system resist combined axial load, moment, and shear. Refer to moment frame.

COLUMN CAPITAL is an enlargement of the top of a concrete column located directly below the slab or drop panel that is cast monolithically with the column.

COMPLIANCE REQUIREMENTS is a construction-related Code requirements directed to the contractor to be incorporated into construction documents by the licensed design professional, as applicable.

COMPOSITE CONCRETE FLEXURAL MEMBERS are concrete flexural members of precast or cast-in-place concrete elements, constructed in separate placements but connected so that all elements respond to loads as a unit.

COMPRESSION-CONTROLLED SECTION is a cross section in which the net tensile strain in the extreme tension reinforcement at nominal strength is less than or equal to the compression-controlled strain limit.

COMPRESSION-CONTROLLED STRAIN LIMIT is a net tensile strain at balanced strain conditions.

CONCRETE are mixture of portland cement or any other hydraulic cement, fine aggregate, coarse aggregate and water, with or without admixtures.

CONCRETE, ALL-LIGHTWEIGHT is a lightweight concrete containing only lightweight coarse and fine aggregates that conform to ASTM C330.

CONCRETE, LIGHTWEIGHT is a concrete containing lightweight aggregate and an equivalent density, as determined by ASTM C567, between 1440 and 1840 kg/m³.

CONCRETE, NON-PRESTRESSED is a reinforced concrete with at least the minimum amount of non-prestressed reinforcement and no prestressed reinforcement; or for two-way slabs, with less than the minimum amount of prestressed reinforcement.

CONCRETE, NORMALWEIGHT is a concrete containing only aggregate that conforms to ASTM C33.

CONCRETE, PLAIN is a concrete with no reinforcement or with less reinforcement than the minimum amount specified for reinforced concrete.

CONCRETE, PRECAST is a concrete element cast elsewhere than its final position in the structure.

CONCRETE, PRESTRESSED is a concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from service loads.

CONCRETE, REINFORCED is a concrete reinforced with at least the minimum amounts of non-prestressed or prestressed reinforcement required by this Code.

CONCRETE, SAND-LIGHTWEIGHT is a concrete containing only normal weight fine aggregate that conforms to ASTM C33M and lightweight coarse aggregate that conforms to ASTM C330M.

CONCRETE, STEEL FIBER-REINFORCED is a concrete containing a prescribed amount of dispersed, randomly oriented, discontinuous deformed steel fibers.

CONCRETE STRENGTH, SPECIFIED COMPRESSIVE is a compressive strength of concrete used in design and evaluated in accordance with provisions of this Code, MPa. Whenever the quantity f'_c is under a radical sign, square root of numerical value only is intended, and result has units of MPa.

CONCRETE BREAKOUT STRENGTH is a strength corresponding to a volume of concrete surrounding the anchor or group of anchors separating from the member.

CONCRETE PRYOUT STRENGTH is a strength corresponding to formation of a concrete spall behind short, stiff anchors displaced in the direction opposite to the applied shear force.

CONNECTION is a region of a structure that joins two or more members; a connection also refers to a region that joins members of which one or more is precast.

CONNECTION, DUCTILE is a connection that experiences yielding as a result of the earthquake design displacements.

CONNECTION, STRONG is a connection between one or more precast elements that remains elastic while adjoining members experience yielding as a result of the earthquake design displacements.

CONTRACT DOCUMENTS is a written and graphic documents and specifications prepared or assembled for describing the location, design, materials, and physical characteristics of the elements of a project necessary for obtaining a building permit and construction of the project.

CONTRACTION JOINT is a formed, sawed, or tooled groove in a concrete structure to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure.

COVER, SPECIFIED CONCRETE is a distance between the outermost surface of embedded reinforcement and the closest outer surface of the concrete.

CROSSTIE is a continuous reinforcing bar having a seismic hook at one end and a hook not less than 90-degree hooks with at least six-diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90-degree hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.

D-REGION is a portion of a member within a distance, h , from a force discontinuity or a geometric discontinuity

DESIGN DISPLACEMENT is a total calculated lateral displacement expected for the design-basis earthquake.

DESIGN INFORMATION is project-specific information to be incorporated into construction documents by the licensed design professional, as applicable.

DESIGN LOAD COMBINATION is a combination of factored loads and forces.

DESIGN STOREY DRIFT RATIO is a relative difference of design displacement in between the top and bottom of a storey, divided by the story height.

DEVELOPMENT LENGTH is a length of embedded reinforcement, including prestressing strand, required to develop the design strength of reinforcement at a critical section.

DISCONTINUITY is an abrupt change in geometry or loading.

DISTANCE SLEEVE is a sleeve that encases the center part of an undercut anchor, a torque-controlled expansion anchor, or a displacement-controlled expansion anchor, but does not expand.

DROP PANEL is a projection below the slab used to reduce the amount of negative reinforcement over a column or the minimum required slab thickness, and to increase the slab shear strength.

DUCT is a conduit, plain or corrugated, to accommodate prestressing reinforcement for post-tensioning applications.

DURABILITY is an ability of a structure or member to resist deterioration that impairs performance or limits service life of the structure in the relevant environment considered in design.

EDGE DISTANCE is a distance from the edge of the concrete surface to the center of the nearest anchor.

EFFECTIVE DEPTH OF SECTION is a distance measured from extreme compression fiber to centroid of tension reinforcement.

EFFECTIVE EMBEDMENT DEPTH is an overall depth through which the anchor transfers force to or from the surrounding concrete; effective embedment depth will normally be the depth of the concrete failure surface in tension applications; for cast-in headed anchor bolts and headed studs, the effective embedment depth is measured from the bearing contact surface of the head.

EFFECTIVE PRESTRESS is a stress remaining in prestressing reinforcement after all losses in Section 420.3.2.6 have occurred.

EMBEDMENTS is an items embedded in concrete, excluding reinforcement as defined in Section 420 and anchors as defined in Section 417. Reinforcement or anchors welded, bolted or otherwise connected to the embedded item to develop the strength of the assembly are considered to be part of the embedment.

EMBEDMENTS, PIPE is an embedded pipes, conduits, and sleeves.

EMBEDMENT LENGTH is a length of embedded reinforcement provided beyond a critical section.

EQUILIBRIUM DENSITY is a density of lightweight concrete determined in accordance with ASTM C 567 after exposure to a relative humidity of 50 ± 5 percent and a temperature of 23 ± 2.00 °C for a period of time sufficient to reach constant density.

EXPANSION SLEEVE is an outer part of an expansion anchor that is forced outward by the center part, either by applied torque or impact, to bear against the sides of the predrilled hole. Refer to anchor, expansion.

EXTREME TENSION REINFORCEMENT is a layer of prestressed or non-prestressed reinforcement that is the farthest from the extreme compression fiber.

FINITE ELEMENT ANALYSIS is a numerical modeling technique in which a structure is divided into a number of discrete elements for analysis.

FIVE PERCENT FRACTILE is a statistical term meaning 90 percent confidence that there is 95 percent probability of the actual strength exceeding the nominal strength.

HEADED DEFORMED BARS is a deformed reinforcing bars with heads attached at one or both ends.

HEADED SHEAR STUD REINFORCEMENT is a reinforcement consisting of individual headed studs or groups of studs, with anchorage provided by a head at each end, or by a head at one end and a common base rail consisting of a steel plate or shape at the other end.

HOOP is a closed tie or continuously wound tie. A closed tie, made up of one or several reinforcement elements, each having seismic hooks at both ends. A closed tie shall not be made up of interlocking headed deformed bars. Section 425.7.4.

INSPECTION is an observation, verification, and required documentation of the materials, installation, fabrication, erection or placement of components and connections to determine compliance with construction documents and referenced standards.

INSPECTION, CONTINUOUS is the full time observation, verification, and required documentation of work in the area where the work is being performed.

INSPECTION, PERIODIC is the part-time or intermittent observation, verification, and required documentation of work in the area where the work is being performed.

ISOLATION JOINT is a separation between adjoining parts of a concrete structure, usually a vertical plane, at a designed location such as to interfere least with performance of the structure, yet such as to allow relative movement in three directions and avoid formation of cracks elsewhere in the concrete and through which all or part of the bonded reinforcement is interrupted.

JACKING FORCE In prestressed concrete, temporary force exerted by device that introduces tension into prestressing reinforcement.

JOINT is a portion of structure common to intersecting members.

LICENSED DESIGN PROFESSIONAL is an individual who is licensed to practice structural design as defined by the statutory requirements of the Professional Regulation Commission (PRC) or jurisdiction in which the project is to be constructed and who is in responsible charge of the structural design.

LOAD are forces or other actions that result from the weight of all building materials, occupants, and their possessions, environmental effects, differential movement, and restrained dimensional changes; permanent loads are those loads in which variations over time are rare or of small magnitude; all other loads are variable loads.

LOAD, DEAD is the weight of the members, supported structure, and permanent attachments or accessories that are likely to be present on a structure in service; or loads meeting specific criteria found in the general building code; without load factors.

LOAD, FACTORED is a load, multiplied by appropriate load factors.

LOAD, LIVE is a load that is not permanently applied to a structure, but is likely to occur during the service life of the structure (excluding environmental loads); or loads meeting specific criteria found in the general building code; without load factors.

LOAD, ROOF LIVE is a load on a roof produced during maintenance by workers, equipment, and materials, and during the life of the structure by movable objects, such as planters or other similar small decorative appurtenances that are not occupancy related; or loads meeting specific criteria found in the general building code; without load factors.

LOAD, SERVICE are all loads, static or transitory, imposed on a structure or element thereof, during the operation of a facility, without load factors.

LOAD PATH are sequence of members and connections designed to transfer the factored loads and forces in such combinations as are stipulated in this Code, from the point of application or origination through the structure to the final support location or the foundation.

MANUFACTURER'S PRINTED INSTALLATION INSTRUCTIONS (MPII) published instructions for the

correct installation of the anchor under all covered installation conditions as supplied in the product packaging.

MODULUS OF ELASTICITY is a ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material.

MOMENT FRAME is a frame in which beams, slabs, columns, and joints resist forces predominantly shear, and axial force; Beams or slabs are predominantly horizontal or nearly horizontal; Columns are predominantly vertical or nearly vertical.

MOMENT FRAME, ORDINARY is a cast-in-place or precast concrete beam-column or slab-column frame complying with Section 418.3.

MOMENT FRAME, INTERMEDIATE is a cast-in-place frame or two-way slab-column frame beams complying with Section 418.4.

MOMENT FRAME, SPECIAL is a cast-in-place beam-column frame complying with Sections 418.2.3 through 418.2.8; and, Sections 418.6 through 418.8. A precast beam-column frame complying with Sections 418.2.3 through 418.2.8 and 418.9.

NET TENSILE STRAIN is a tensile strain at nominal strength exclusive of strains due to effective prestress, creep, shrinkage and temperature.

NODAL ZONE is a volume of concrete around a node that is assumed to transfer strut-and-tie forces through the node.

NODE is a point in a strut-and-tie model where the axes of the struts, ties, and concentrated forces acting on the joint intersect.

ONE-WAY CONSTRUCTION is members designed to be capable of supporting all loads through bending in a single direction; refer to two-way construction.

PEDESTAL is a member with a ratio of height-to-least lateral dimension less than or equal to three used primarily to support axial compressive load; for a tapered member, the least lateral dimension is the average of the top and bottom dimensions of the smaller side.

PLASTIC HINGE REGION is a length of frame element over which flexural yielding is intended to occur due to earthquake design displacements, extending not less than a distance h from the critical section where flexural yielding initiates.

POST-TENSIONING is a method of prestressing in which prestressing reinforcement is tensioned after concrete has hardened.

PRECOMPRESSED TENSILE ZONE is a portion of a prestressed member where flexural tension, calculated using gross section properties, would occur under service loads if the prestress force was not present.

PRETENSIONING is a method of prestressing in which prestressing reinforcement is tensioned before concrete is cast.

PROJECTED AREA is an area on the free surface of the concrete member that is used to represent the larger base of the assumed rectilinear failure surface.

PROJECTED INFLUENCE AREA is a rectilinear area on the free surface of the concrete member that is used to calculate the bond strength of adhesive anchors.

REINFORCEMENT is a steel element or elements embedded in concrete and conforming to Sections 420.2 through 420.5. Prestressed reinforcement in external tendons is also considered reinforcement.

REINFORCEMENT, DEFORMED is a deformed bars, welded bar mats, deformed wire, and welded wire reinforcement conforming to Sections 420.2.1.3, 420.2.1.5, or 420.2.1.7, excluding plain wire.

REINFORCEMENT, PLAIN are bars or wires conforming to Section 420.2.1.4 or 420.2.1.7 that do not conform to definition of deformed reinforcement.

REINFORCEMENT, PRESTRESSED is a prestressing reinforcement that has been tensioned to impart forces to concrete.

REINFORCEMENT, BONDED PRESTRESSED is a pretensioned reinforcement or prestressed reinforcement in a bonded tendon.

REINFORCEMENT, PRESTRESSING is a high-strength reinforcement such as strand, wire, or bar conforming to Section 420.3.1.

REINFORCEMENT, WELDED WIRE is a plain or deformed wire fabricated into sheets or rolls conforming to Section 420.2.1.7.

REINFORCEMENT, WELDED DEFORMED STEEL BAR MAT is a mat conforming to Section 420.2.1.5 consisting of two layers of deformed bars at right angles to each other welded at the intersections.

REINFORCEMENT, ANCHOR is an reinforcement used to transfer the full design load from the anchors into the structural member.

REINFORCEMENT, SUPPLEMENTARY is a reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load from the anchors into the structural member.

SEISMIC DESIGN CATEGORY is a classification assigned to a structure based on its occupancy category and the severity of the design earthquake ground motion at the site, as defined in Section 402 of this Code.

SEISMIC-FORCE-RESISTING SYSTEM is a portion of the structure designed to resist earthquake effects required by the legally adopted general building code using the applicable provisions and load combinations.

SEISMIC HOOK is a hook on a stirrup, or crosstie having a bend not less than 135 degrees, except that circular hoops shall have a bend not less than 90 degrees; hooks shall have a $6d_b$, but not less than 75 mm. The hooks shall engage the longitudinal reinforcement and the extension shall project into the interior of the stirrup or hoop.

SHEAR CAP is a projection below the slab used to increase the slab shear strength.

SHEATHING is a material encasing prestressing reinforcement to prevent bonding of the prestressing reinforcement with the surrounding concrete, to provide corrosion protection, and to contain the corrosion inhibiting coating.

SPACING is a center-to-center distance between adjacent items, such as longitudinal reinforcement, transverse reinforcement, prestressing reinforcement, or anchors.

SPACING, CLEAR is a least dimension between the outermost surfaces of adjacent items.

SPAN LENGTH is a distance between supports.

SPECIAL SEISMIC SYSTEMS is a structural systems that use special moment frames, special structural walls, or both.

SPECIALTY INSERT is a predesigned and prefabricated cast-in anchors specifically designed for attachment of bolted or slotted connections.

SPIRAL REINFORCEMENT is a continuously wound reinforcement in the form of a cylindrical helix.

SPLITTING TENSILE STRENGTH is a tensile strength of concrete determined in accordance with ASTM C496M as described in "Specifications for Lightweight Aggregate for Structural Concrete" (ASTM C330).

STEEL ELEMENT, BRITTLE is an element with a tensile test elongation of less than 14 percent, or reduction in area of less than 30 percent at failure.

STEEL ELEMENT, DUCTILE is an element with a tensile test elongation of at least 14 percent and reduction in area of at least 30 percent; Steel element meeting the requirements of ASTM A307 shall be considered ductile; except as modified by for earthquake effects, deformed reinforcing bars meeting the requirements of ASTM A615M, A706M, or A955M shall be considered as ductile steel elements.

STIRRUP is an reinforcement used to resist shear and torsion stresses in a structural member; typically deformed bars, deformed wires, or welded wire reinforcement either single leg or bent into **L**, **U** or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. Refer to "tie."

STRENGTH, DESIGN is a nominal strength multiplied by a strength reduction factor, ϕ .

STRENGTH, NOMINAL is strength of a member or cross section calculated in accordance with provisions and assumptions of the strength design method of this chapter before application of any strength reduction factors.

STRENGTH, REQUIRED is strength of a member or cross section required to resist factored loads or related internal moments and forces in such combinations as are stipulated in this chapter.

STRESS is the intensity of force per unit area.

STRESS LENGTH is a length of anchor, extending beyond concrete in which it is anchored, subject to full tensile load applied to anchor, and for which cross-sectional area is minimum and constant.

STRUCTURAL CONCRETE is a concrete used for structural purposes, including plain and reinforced concrete.

STRUCTURAL DIAPHRAGM is a member, such as a floor or roof slab, that transmits forces acting in the plane of the member to the vertical elements of the seismic-force-resisting system. A structural diaphragm may include chords and collectors as part of the diaphragm.

STRUCTURAL INTEGRITY is an ability of a structure through strength, redundancy, ductility, and detailing of reinforcement to redistribute stresses and maintain overall stability if localized damage or significant overstress occurs.

STRUCTURAL SYSTEM interconnected members designed to meet performance requirements.

STRUCTURAL TRUSS is an assemblage of reinforced concrete members subjected primarily to axial forces.

STRUCTURAL WALL is a wall proportioned to resist combinations of shears, moments, and axial forces in the plane of the wall; a shear wall is a structural wall.

STRUCTURAL WALL, ORDINARY REINFORCED CONCRETE is a wall complying with the requirements of Sections 411.

STRUCTURAL WALL, ORDINARY PLAIN CONCRETE is a wall complying with the requirements of Section 414.

STRUCTURAL WALL, INTERMEDIATE PRECAST is a wall complying with Section 418.5.

STRUCTURAL WALL, SPECIAL is a cast-in-place structural wall in accordance with Sections 418.2.3 through 418.2.8 and 418.10; or precast structural wall in accordance with Sections 418.2.3 through 418.2.8 and 418.11.

STRUT is a compression member in a strut-and-tie model representing the resultant of a parallel or a fan-shaped compression field.

STRUT, BOTTLE-SHAPED is a strut that is wider at mid-length than at its ends.

STRUT-AND-TIE MODEL is a truss model of a structural member or of a D-region in such a member, made up of struts and ties connected at nodes, capable of transferring the factored loads to the supports or to adjacent B-regions.

TENDON In post-tensioned members, a tendon is a complete assembly consisting of anchorages, prestressing reinforcement, and sheathing with coating for unbonded applications or ducts filled with grout for bonded applications.

TENDON, BONDED is a tendon in which prestressed reinforcement is continuously bonded to the concrete through grouting of ducts embedded within the concrete cross section.

TENDON, EXTERNAL is a tendon external to the member concrete cross section in post-tensioned applications.

TENDON, UNBONDED is a tendon in which prestressed reinforcement is prevented from bonding to the concrete. The prestressing force is permanently transferred to the concrete at the tendon ends by the anchorages only.

TENSION-CONTROLLED SECTION is a cross section in which the net tensile strain in the extreme tension steel at nominal strength is greater than or equal to 0.005.

TIE is a loop of reinforcing bar or wire enclosing longitudinal reinforcement; a continuously wound bar or wire in the form of a circle, rectangle, or other polygon shape without re-entrant corners is acceptable; refer to stirrup or hoop; or a tension member in a strut-and-tie model.

TRANSFER is an act of transferring stress in prestressing reinforcement from jacks or pretensioning bed to concrete member.

TRANSFER LENGTH is a length of embedded pretensioned reinforcement required to transfer the effective prestress to the concrete.

TWO-WAY CONSTRUCTION are members designed to be capable of supporting loads through bending in two directions; Some slabs and foundations are considered two-way construction. Refer to one-way construction.

WALL is a vertical element designed to resist axial load, lateral load, or both, with a horizontal length-to-thickness ratio greater than three, used to enclose or separate spaces.

WALL SEGMENT is a portion of wall bounded by vertical or horizontal openings or edges.

WALL SEGMENT, HORIZONTAL is a segment of a structural wall, bounded vertically by two openings or by an opening and an edge.

WALL SEGMENT, VERTICAL is a segment of a structural wall, bounded horizontally by two openings or by an opening and an edge; Wall piers are vertical wall segments.

WALL PIER is a vertical wall segments with dimensions and reinforcement intended to result in shear demand being limited by flexural yielding of the vertical reinforcement in the pier.

WATER-CEMENTITIOUS MATERIALS RATIO is a ratio of mass of water, excluding that absorbed by the aggregate, to the mass of cementitious materials in a mixture, stated as a decimal.

WORK is the entire construction or separately identifiable parts thereof that are required to be furnished under the construction documents.

YIELD STRENGTH is a specified minimum yield strength or yield point of reinforcement; yield strength or yield point shall be determined in tension according to applicable ASTM standards as modified by this section.

SECTION 403 REFERENCED STANDARDS

403.1 Scope

403.1 Standards, or specific sections thereof, cited in this Code, including Annex, Appendices, or Supplements where prescribed, are referenced without exception in this Code, unless specifically noted. Cited standards are listed in the following with their serial designations, including year of adoption or revision.

403.2 Referenced Standards

403.2.1 American Association of State Highway and Transportation Officials (AASHTO)

Article Designation	Title
Article 5.10.9.6 Article 5.10.9.7.2 Article 5.10.9.7.3	AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
Article 10.3.2.3	AASHTO LRFD Bridge Construction Specifications, 3rd Edition, 2010

403.2.2 American Concrete Institute (ACI)

Standard/ Section Designation	Title
Section 4.2.3 of ACI 301-10	Specifications for Structural Concrete
ACI 318.2-14	Building Code Requirements for Concrete Thin Shells
ACI 332-14	Residential Code Requirements for Structural Concrete and Commentary
ACI 355.2-07	Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary
ACI 355.4-11	Qualification of Post-Installed Adhesive Anchors in Concrete
ACI 374.1-05	Acceptance Criteria for Moment Frames Based on Structural Testing
ACI 423.7-07	Specification for Unbonded Single-Strand Tendon Materials
ACI ITG-5.1-07	Acceptance Criteria for Special Unbonded Post-Tensioned Precast Structural Walls Based on Validation Testing

403.2.3 American Society of Civil Engineers (ASCE)/Structural Engineering Institute (SEI)

Section Designation	Title
Section 2.3.3, Load Combinations including Flood Loads	ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures
Section 2.3.4, Load Combinations excluding Atmospheric Ice Loads	

403.2.4 ASTM International

Standard Designation	Title
A36/A36M-12	Standard Specification for Carbon Structural Steel
A53/A53M-12	Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless
A184/A184M-06 (2011)	Standard Specification for Welded Deformed Steel Bar Mats for Concrete Reinforcement
A242/A242M-04 (2009)	Standard Specification for High-Strength Low-Alloy Structural Steel
A307-12	Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod, 420 MPa Tensile Strength
A416/A416M-12a	Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
A421/A421M-10	Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete including Supplementary Requirement I, Low-Relaxation Wire
A500/A500M-10a	Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
A501-07	Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
A572/A572M-12a	Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel

Standard Designation	Title
A588/A588M-10	Standard Specification for High-Strength Low-Alloy Structural Steel, up to 345 MPa Minimum Yield Point, with Atmospheric Corrosion Resistance
A615/A615M-12	Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
A706/A706M-09b	Steel Deformed and Plain Bars for Concrete Reinforcement
A722/A722M-12	Standard Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete
A767/A767M-09	Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement
A775/A775M-07b	Standard Specification for Epoxy-Coated Steel Reinforcing Bars
A820/A820M-11	Standard Specification for Steel Fibers for Fiber-Reinforced Concrete
A884/A884M-12	Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement
A934/A934M-07	Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars
A955/A955M-12 ^{c1}	Standard Specification for Deformed and Plain Stainless-Steel Bars for Concrete Reinforcement
A970/A970M-12	Standard Specification for Headed Steel Bars for Concrete Reinforcement including Annex A1 Requirements or Class HA Headed Dimensions
A992/A992M-11	Standard Specification for Structural Steel Shapes
A996/A996M-09b	Standard Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement
A1022/A1022M-07	Standard Specification for Deformed and Plain Stainless Steel Wire and Welded Wire for Concrete Reinforcement

Standard Designation	Title
A1035/A1035M-11	Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement
A1044/A1044M-05 (2010)	Standard Specification for Steel Stud Assemblies for Shear Reinforcement of Concrete
A1055/A1055M-10 ^{c1}	Standard Specification for Zinc and Epoxy Dual-Coated Steel Reinforcing Bars
A1060/A1060M-11 ^{c1}	Standard Specification for Zinc-Coated (Galvanized) Steel Welded Wire Reinforcement, Plain and Deformed, for Concrete
A1064/A1064M-12	Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete
C29/C29M-09	Standard Test Method for Bulk Density("Unit Weight") and Voids in Aggregate
C31/C31M-12	Standard Practice for Making and Curing Concrete Test Specimens in the Field
C33/C33M-13	Standard Specification for Concrete Aggregates
C39/C39M-12a	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C42/C42M-13	Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C94/C94M-12a	Standard Specification for Ready-Mixed Concrete
C144-11	Standard Specification for Aggregate for Masonry Mortar
C150/C150M-12	Standard Specification for Portland Cement
C172/C172M-10	Standard Practice for Sampling Freshly Mixed Concrete
C173/173M-12	Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method
C231/C231M-10	Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method
C260/C260M-10a	Standard Specification for Air-Entraining Admixtures For Concrete

Standard Designation	Title
C330/C330M-09	Standard Specification for Lightweight Aggregates for Structural Concrete
C496/C496M-11	Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C567/C567M-11	Standard Test Method or Determining Density of Structural Lightweight Concrete
C595/C595M-12 ^{c1}	Standard Specification for Blended Hydraulic Cements
C618-12a	Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
C685/C685M-11	Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing
C845/C845M-12	Standard Specification for Expansive Hydraulic Cement
C989/C989M-12a	Standard Specification for Slag Cement for Use in Concrete and Mortars
C1012/C1012M-13	Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution
C1017/C1017M-07	Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete
C1077-13	Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation
C1116/C116M-10a	Standard Specification for Fiber-Reinforced Concrete
C1157/C1157M-11	Standard Performance Specification for Hydraulic Cement
C1218/1218M-99(2008)	Standard Test Method for Water-Soluble Chloride in Mortar and Concrete
C1240-12	Standard Specification for Silica Fume Used in Cementitious Mixtures
C1580-09 ^{c1}	Standard Test for Water-Soluble Sulfate in Soil
C1582/C1582M-11	Standard Specification for Admixtures to Inhibit Chloride-Induced Corrosion of Reinforcing Steel in Concrete

Standard Designation	Title
C1602/C1602M-12	Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete
C1609/C1609M-12	Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third- Point Loading)
D516-11	Standard Test Method for Sulfate Ion in Water
D4130-08	Standard Test Method for Sulfate Ion in Brackish Water, Seawater, and Brines

403.2.5 American Welding Society (AWS)

Standard Designation	Title
A WS D1.4/D1.4M: 2011	Structural Welding Code Reinforcing Steel
AWS D1.1/D1.1M: 2010	Structural Welding Code Steel

403.2.6 Australian Standard (AS) and New Zealand Standard (NZS)

Standard Designation	Title
AS/NZS 4671: 2001	Steel Reinforcing Material
NZS 3101: 2006 Part 1 & Part 2	Concrete Structures Standard Design of Concrete Structures
NZS 3109 Amendment 2	Welding of Reinforcing Steel
AS/NZS 1554.3: 2008 Part 3	Structural Steel Welding-Welding of Reinforcing Steel

SECTION 404 STRUCTURAL SYSTEM REQUIREMENTS

404.1 Scope

404.1.1 This section shall apply to design of structural concrete in structures or portions of structures defined in Section 401.

404.2 Materials

404.2.1 Design properties of concrete shall be selected to be in accordance with Section 419.

404.2.2 Design properties of reinforcement shall be selected to be in accordance with Section 420.

404.3 Design Loads

404.3.1 Loads and load combinations considered in design shall be in accordance with Section 405.

404.4 Structural System and Load Paths

404.4.1 The structural system shall include (a) through (g), as applicable:

- Floor construction and roof construction, including one-way and two-way slabs;
- Beams and joists;
- Column;
- Wall;
- Diaphragms;
- Foundations;
- Joints, connections, and anchors as required to transmit forces from one component to another.

404.4.2 Design of structural members including joints and connections given in Section 404.4.1 shall be in accordance with Sections 407 through 418.

404.4.3 It shall be permitted to design a structural system comprising structural members not in accordance with Sections 404.4.1 and 404.4.2, provided the structural system is approved in accordance with Section 401.10.1.

404.4.4 The structural system shall be designed to resist the factored loads in load combinations given in Section 404.3 without exceeding the appropriate member design strengths, considering one or more continuous load paths from the point of load application or origination to the final point of resistance.

404.4.5 Structural systems shall be designed to accommodate anticipated volume change and differential settlement.

404.4.6 Seismic-Force-Resisting System

404.4.6.1 Every structure shall be assigned to a seismic zones 4, or 2, in accordance with the general building code or as determined by the authority having jurisdiction in areas without a legally adopted building code.

404.4.6.2 Structural systems designated as part of the seismic-force-resisting system shall be restricted to those systems designated by the general building code or as determined by the authority having jurisdiction in areas without a legally adopted building code.

404.4.6.3 Structural systems assigned to seismic zone 2 shall satisfy the applicable requirements of this Code. Structures assigned to seismic zone 2 are not required to be designed in accordance with Section 418.

404.4.6.4 Structural systems assigned to seismic zone 4 shall satisfy the requirements of Section 418 in addition to applicable requirements of other sections of this Code.

404.4.6.5 In structures assigned to seismic zone 4, structural members assumed not to be part of the seismic-force-resisting system shall be permitted, provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of damage to structural and nonstructural members that are not a part of the seismic-force-resisting system shall be considered.

404.4.7 Diaphragms

404.4.7.1 Diaphragms, such as floor or roof slabs, shall be designed to resist simultaneously both out-of-plane gravity loads and in-plane lateral forces in load combinations given in Section 404.3.

404.4.7.2 Diaphragms and their connections to framing members shall be designed to transfer forces between the diaphragm and framing members.

404.4.7.3 Diaphragms and their connections shall be designed to provide lateral support to vertical, horizontal, and inclined elements.

404.4.7.4 Diaphragms shall be designed to resist applicable lateral loads from soil and hydrostatic pressure and other loads assigned to the diaphragm by structural analysis.

404.4.7.5 Collectors shall be provided where required to transmit forces between diaphragms and vertical elements.

404.4.7.6 Diaphragms that are part of the seismic-force-resisting system shall be designed for the applied forces. In structures assigned to seismic zone 4, the diaphragm design shall be in accordance with Section 418.

404.5 Structural Analysis

404.5.1 Analytical procedures shall satisfy compatibility of deformations and equilibrium of forces.

404.5.2 The methods of analyses given in Section 406 shall be permitted.

404.6 Strength

404.6.1 Design strength of a member and its joints and connections, in terms of moment, axial force, shear, torsion, and bearing, shall be taken as the nominal strength S_n , multiplied by the applicable strength reduction factor ϕ .

404.6.2 Structures and structural members shall have design strength at all sections, ϕS_n , greater than or equal to the required strength U calculated for the factored loads and forces in such combinations as required by this Section or the general building code.

404.7 Serviceability

404.7.1 Evaluation of performance at service load conditions shall consider reactions, moments, torsions, shears, and axial forces induced by prestressing, creep, shrinkage, temperature change, axial deformation, restraint of attached structural members, and foundation settlement.

404.7.2 For structures, structural members, and their connections, the requirements of Section 404.7.1 shall be deemed to be satisfied if designed in accordance with the provisions of the applicable member sections.

404.8 Durability

404.8.1 Concrete mixtures shall be designed in accordance with the requirements of Sections 419.3.2 and 426.4, considering applicable environmental exposure to provide required durability.

404.8.2 Reinforcement shall be protected from corrosion in accordance with Section 420.6.

404.9 Sustainability

404.9.1 The licensed design professional shall be permitted to specify in the construction documents sustainability requirements in addition to strength, serviceability, and durability requirements of this Code.

404.9.2 The strength, serviceability, and durability requirements of this Code shall take precedence over sustainability considerations.

404.10 Structural Integrity

404.10.1 General

404.10.1.1 Reinforcement and connections shall be detailed to tie the structure together effectively and to improve overall structural integrity.

404.10.2 Minimum Requirements for Structural Integrity

404.10.2.1 Structural members and their connections shall be in accordance with structural integrity requirements in Table 404.10.2.1.

Table 404.10.2.1
Minimum Requirements for Structural Integrity

Member Type	Section
Non-prestressed two-way slabs	408.7.4.2
Prestressed two-way slabs	408.7.5.6
Non-prestressed two-way joist systems	408.8.1.6
Cast-in-place beams	409.7.7
Non-prestressed one-way joist systems	409.8.1.6
Precast joints and connections	416.2.1.8

404.11 Fire Resistance

404.11.1 Structural concrete members shall satisfy the fire protection requirements of the general building code.

404.11.2 Where the general building code requires a thickness of concrete cover for fire protection greater than the concrete cover specified in Section 420.6.1, such greater thickness shall govern.

404.12 Requirements for Specific Types of Construction

404.12.1 Precast Concrete Systems

404.12.1 Design of precast concrete members and connections shall include loading and restraint conditions from initial fabrication to end use in the structure, including form removal, storage, transportation, and erection.

404.12.1.2 Design, fabrication, and construction of precast members and their connections shall include the effects of tolerances.

404.12.1.3 When precast members are incorporated into a structural system, the forces and deformations occurring in and adjacent to connections shall be included in the design.

404.12.1.4 Where system behavior requires in-plane loads to be transferred between the members of a precast floor or wall system, (a) and (b) shall be satisfied:

- a. In-plane load paths shall be continuous through both connections and members.
- b. Where tension loads occur, a load path of steel or steel reinforcement, with or without splices, shall be provided.

404.12.1.5 Distribution of forces that act perpendicular to the plane of precast members shall be established by analysis or test.

404.12.2 Prestressed Concrete Systems

404.12.2.1 Design of prestressed members and systems shall be based on strength and on behavior at service conditions at all critical stages during the life of the structure from the time prestress is first applied.

404.12.2.2 Provisions shall be made for effects on adjoining construction of elastic and plastic deformations, deflections, changes in length, and rotations due to prestressing. Effects of temperature change, restraint of attached structural members, foundation settlement, creep, and shrinkage shall also be considered.

404.12.2.3 Stress concentrations due to prestressing shall be considered in design.

404.12.2.4 Effect of loss of area due to open ducts shall be considered in computing section properties before grout in post-tensioning ducts has attained design strength.

404.12.2.5 Post-tensioning tendons shall be permitted to be external to any concrete section of a member. Strength and serviceability design requirements of this Code shall be used to evaluate the effects of external tendon forces on the concrete structure.

404.12.3 Composite Concrete Flexural Members

404.12.3.1 This Chapter shall apply to composite concrete flexural members as defined in Section 402.

404.12.3.2 Individual members shall be designed for all critical stages of loading.

404.12.3.3 Members shall be designed to support all loads introduced prior to full development of design strength of composite members.

404.12.3.4 Reinforcement shall be detailed to minimize cracking and to prevent separation of individual components of composite members.

404.12.4 Composite Steel and Concrete Construction

404.12.4.1 Composite compression members shall include all members reinforced longitudinally with structural steel shapes, pipe, or tubing with or without longitudinal bars.

404.12.4.1 The design of composite compression members shall be in accordance with Section 410.

404.12.5 Structural Plain Concrete Systems

404.12.5.1 The design of structural plain concrete members, both cast-in-place and precast, shall be in accordance with Section 414.

404.13 Construction and Inspection

404.13.1 Specifications for construction execution shall be in accordance with Section 426.

404.13.2 Specifications for construction execution shall be in accordance with Section 426.

404.14 Strength Evaluation of Existing Structures

404.14.1 Strength evaluation of existing structures shall be in accordance with Section 427.

SECTION 405 LOADS

405.1 Scope

405.1.1 This Section shall apply to selection of load factors and combinations used in design, except as permitted in Section 427.

405.2 General

405.2.1 Loads shall include self-weight; applied loads; and effects of prestressing, earthquakes, restraint of volume change, and differential settlement.

405.2.2 Loads and seismic zones shall be in accordance with the general building code, or determined by another authority having jurisdiction in areas without a legally adopted building code.

405.2.3 Live load reductions shall be permitted in accordance with the general building code or, in the absence of a general building code, in accordance with ASCE/SEI 7.

405.3 Load Factors and Combinations

405.3.1 Required strength, U , shall be at least equal to the effects of factored loads in Table 405.3.1, with exceptions and additions in Sections 405.3.3 through 405.3.12.

Table 405.3.1 Load Combinations

Load Designation	Equation	Primary Load
$U = 1.4D$	(405.31a)	D
$U = 1.2D + 1.6L + 0.5(L_r \text{ or } R)$	(405.31b)	L
$U = 1.2D + 1.6(L_r \text{ or } R) + (1.0L \text{ or } 0.5W)$	(405.31c)	$L_r \text{ or } R$
$U = 1.2D + 1.0W + 1.0L + 0.5(L_r \text{ or } R)$	(405.31d)	W
$U = 1.2D + 1.0E + 1.0L$	(405.31e)	E
$U = 0.9D + 1.0W$	(405.31f)	W
$U = 0.9D + 1.0E$	(405.31g)	E

405.3.2 The effect of one or more loads not acting simultaneously shall be investigated.

405.3.3 The load factor on live load L in Eqs. 405.3.1c, 405.3.1d, and 405.3.1e shall be permitted to be reduced to 0.5 except for (a), (b), or (c):

- Garages;
- Areas occupied as places of public assembly;

- c. Areas where L is greater than 4.8 kPa.

405.3.4 If applicable, L shall include (a) through (f):

- a. Concentrated live loads;
- b. Vehicular loads;
- c. Crane loads;
- d. Loads on hand rails, guardrails, and vehicular barrier systems;
- e. Impact effects;
- f. Vibration effects.

405.3.5 If wind load W is based on service-level loads, $1.6W$ shall be used in place of $1.0W$ in Eqs. 405.3.1d and 405.3.1f, and $0.8W$ shall be used in place of $0.5W$ in Eq. 405.3.1c.

405.3.6 The structural effects of forces due to restraint of volume change and differential settlement, T , shall be considered in combination with other loads if the effects of T can adversely affect structural safety or performance. The load factor for T shall be established considering the uncertainty associated with the likely magnitude of T , the probability that the maximum effect of T will occur simultaneously with other applied loads, and the potential adverse consequences if the effect of T is greater than assumed. The load factor on T shall not have a value less than 1.0.

405.3.7 If fluid load F is present, it shall be included in the load combination equations of Section 405.3.1 in accordance with (a), (b), (c) or (d):

- a. If F acts alone or adds to the effects of D , it shall be included with a load factor of 1.4 in Eq. 405.3.1a;
- b. If F adds to the primary load, it shall be included with a load factor of 1.2 in Eq. 405.3.1b through 405.3.1e;
- c. If F adds to the primary load, it shall be included with a load factor of 1.2 in Eq. 405.3.1b through 405.3.1e;
- d. If the effect of F is permanent and counteracts the primary load, it shall be included with a load factor of 0.9 in Eq. 405.3.1g.
- e. If the effect of F is not permanent but, when present, counteracts the primary load, F shall not be included in Eq. 405.3.1a through 405.3.1g.

405.3.8 If lateral earth pressure H is present, it shall be included in the load combination equations of Section 405.3.1 in accordance with (a), (b), or (c):

- a. If H acts alone or adds to the primary load, it shall be included with a load factor of 1.6 in Eq. 405.3.1a through 405.3.1e;
- b. If the effect of H is permanent and counteracts the primary load, it shall be included with a load factor of 0.9 in Eq. 405.3.1f and 405.3.1g;
- c. If the effect of H is not permanent but, when present, counteracts the primary load, H shall not be included in Eq. 405.3.1a through 405.3.1g.

405.3.9 If a structure is in a flood zone, the flood loads and the appropriate load factors and combinations of ASCE/SEI 7 shall be used.

405.3.10 Required strength U shall include internal load effects due to reactions induced by prestressing with a load factor of 1.0.

405.3.11 For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum prestressing reinforcement jacking force.

SECTION 406 STRUCTURAL ANALYSIS

406.1 Scope

406.1.1 This section shall apply to methods of analysis, modeling of members and structural systems, and calculation of load effects.

406.2 General

406.2.1 Members and structural systems shall be permitted to be modeled in accordance with Section 406.3.

406.2.2 All members and structural systems shall be analyzed for the maximum effects of loads including the arrangements of live load in accordance with Section 406.4.

406.2.3 Methods of analysis permitted by this Section shall be (a) through (e):

- a. The simplified method for analysis of continuous beams and one-way slabs for gravity loads in Section 406.5;
- b. First-order in Section 406.6;
- c. Elastic second-order in Section 406.7 (d) inelastic second-orders in Section 406.8 (e) Finite element in Section 406.9;
- d. Inelastic second-order in Section 406.8;
- e. Finite element in Section 406.9.

406.2.4 Additional analysis methods that are permitted include Sections 406.2.4.1 through 406.2.4.4.

406.2.4.1 Two-way slabs shall be permitted to be analyzed for gravity loads in accordance with (a) or (b):

- a. Direct design method in Section 408.10;
- b. Equivalent frame method in Section 408.11.

406.2.4.2 Slender walls shall be permitted to be analyzed in accordance with Section 411.8 for out-of-plane effects.

406.2.4.3 Diaphragms shall be permitted to be analyzed in accordance with Section 412.4.2.

406.2.4.4 A member or region shall be permitted to be analyzed and designed using the strut-and-tie method in accordance with Section 423.

406.2.5 Slenderness effects shall be permitted to be neglected if (a) or (b) is satisfied:

- a. For columns not braced against sidesway

$$\frac{k\ell_u}{r} \leq 22 \quad (406.2.5a)$$

- b. For columns braced against sidesway

$$\frac{k\ell_u}{r} \leq 34 + 12 M_1/M_2 \quad (406.2.5b)$$

and

$$\frac{k\ell_u}{r} \leq 40 \quad (406.2.5c)$$

where M_1/M_2 is negative if the column is bent in single curvature, and positive for double curvature.

If bracing elements resisting lateral movement of a storey have a total stiffness of at least 12 times the gross lateral stiffness of the columns in the direction considered, it shall be permitted to consider columns within the storey to be braced against sidesway.

406.2.5.1 The radius of gyration, r , shall be permitted to be calculated by (a),(b),or (c):

$$a. \quad r = \sqrt{\frac{I_g}{A_g}} \quad (406.2.5.1)$$

- b. 0.30 times the dimension in the direction stability is being considered for rectangular columns;

- c. 0.25 times the diameter of circular columns.

406.2.5.2 For composite columns, the radius of gyration, r , shall not be taken greater than:

$$r = \sqrt{\frac{(E_c I_g / 5) E_s I_{sx}}{(E_c A_g / 5) E_s A_{sx}}} \quad (406.2.5.2)$$

Longitudinal bars located within a concrete core encased by structural steel or within transverse reinforcement

surrounding a structural steel core shall be permitted to be used in calculating A_{SX} and I_{SX} .

406.2.6 Unless slenderness effects are neglected as permitted by Section 406.2.5, the design of columns, restraining beams, and other supporting members shall be based on the factored forces and moments considering second order effects in accordance with Sections 406.6.4, 406.7, or 406.8. M_u including second-order effects shall not exceed $1.4M_u$ due to first-order effects.

406.3 Modeling Assumptions

406.3.1 General

406.3.1.1 Relative stiffnesses of members within structural systems shall be based on reasonable and consistent assumptions.

406.3.1.2 To calculate moments and shears caused by gravity loads in columns, beams, and slabs, it shall be permitted to use a model limited to the members in the level being considered and the columns above and below that level. It shall be permitted to assume far ends of columns built integrally with the structure to be fixed.

406.3.1.3 The analysis model shall consider the effects of variation of member cross-sectional properties, such as that due to haunches.

406.3.2 T-beam Geometry

406.3.2.1 For non-prestressed T-beams supporting monolithic or composite slabs, the effective flange width b_f shall include the beam web width b_w plus an effective overhanging flange width in accordance with Table 406.3.2.1, where h is the slab thickness and s_w is the clear distance to the adjacent web.

Table 406.3.2.1: Dimensional Limits for Effective Overhanging Flange Width for T-Beams

Flange location	Effective overhanging flange width, beyond face of web	
Each side of web	Least of:	$8h$
		$s_w/2$
		$\ell_n/8$
One side of web	Least of:	$6h$
		$s_w/2$
		$\ell_n/12$

406.3.2.2 Isolated non-prestressed T-beams in which the flange is used to provide additional compression area shall have a flange thickness greater than or equal to

$0.5b_w$ and an effective flange width less than or equal to $4b_w$.

406.3.2.3 For prestressed T-beams, it shall be permitted to use the geometry provided by Sections 406.3.2.1 and 406.3.2.2.

406.4 Arrangement of Live Load

406.4.1 For the design of floors or roofs to resist gravity loads, it shall be permitted to assume that live load is applied only to the level under consideration.

406.4.2 For one-way slabs and beams, it shall be permitted to assume (a) and (b):

- Maximum positive M_u near midspan occurs with factored L on the span and on alternate spans;
- Maximum negative M_u at a support occurs with factored L on adjacent spans only.

406.4.3 For two-way slab systems, factored moments shall be calculated in accordance with Section 406.4.3.1, 406.4.3.2, or 406.4.3.3, and shall be at least the moments resulting from factored L applied simultaneously to all panels.

406.4.3.1 If the arrangement of L is known, the slab system shall be analyzed for that arrangement.

406.4.3.2 If L is variable and does not exceed $0.75D$, or the nature of L is such that all panels will be loaded simultaneously, it shall be permitted to assume that maximum M_u at all sections occurs with factored L applied simultaneously to all panels.

406.4.3.3 For loading conditions other than those defined in Section 406.4.3.1 or 406.4.3.2, it shall be permitted to assume (a) and (b):

- Maximum positive M_u near midspan of panel occurs with 75 percent of factored L on the panel and alternate panels;
- Maximum negative M_u at a support occurs with 75 percent of factored L on adjacent panels only.

406.5 Simplified Method of Analysis for Non-Prestressed Continuous Beams and One-way Slabs

406.5.1 It shall be permitted to calculate M_u and V_u due to gravity loads in accordance with this section for

continuous beams and one-way slabs satisfying (a) through (e):

- Members are prismatic;
- Loads are uniformly distributed;
- $L \leq 3D$;
- There are at least two spans;
- The longer of two adjacent spans does not exceed the shorter by more than 20 percent.

406.5.2 M_u due to gravity loads shall be calculated in accordance with Table 406.5.2.

Table 406.5.2 Approximate Moments for Non-Prestressed Continuous Beams and One-Way Slabs

Moment	Location	Condition	M_u
Positive	End span	Discontinuous end integral with support	$w_u \ell_n^2 / 14$
		Discontinuous end unrestrained	$w_u \ell_n^2 / 11$
	Interior spans	All	$w_u \ell_n^2 / 16$
Negative ^[1]	Interior face of exterior support	Member built integrally with supporting spandrel beam	$w_u \ell_n^2 / 24$
		Member built integrally with supporting column	$w_u \ell_n^2 / 16$
	Exterior face of first interior support	Two spans	$w_u \ell_n^2 / 9$
		More than two spans	$w_u \ell_n^2 / 10$
	Face of other supports	All	$w_u \ell_n^2 / 11$
	Face of all supports satisfying (a) or (b)	a) Slabs with spans not exceeding 3m. b) Beams where ratio of sum of column stiffnesses to beam stiffness exceeds 8 at each end of span	$w_u \ell_n^2 / 12$

^[1] To calculate negative moments, ℓ_n shall be the average of the adjacent clear span lengths.

406.5.3 It Moments calculated in accordance with Section 406.5.2 shall not be redistributed.

406.5.4 V_u due to gravity loads shall be calculated in accordance with Table 406.5.4.

Table 406.5.4 Approximate Shears for Non-Prestressed Continuous Beams and One-Way Slabs

Location	V_u
Exterior face of first interior support	$1.15w_u \ell_n / 2$
Face of all other supports	$w_u \ell_n / 2$

406.5.5 Floor or roof level moments shall be resisted by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffnesses considering conditions of restraint.

406.6 First-order Analysis

406.6.1 General

406.6.1.1 Slenderness effects shall be considered in accordance with Section 406.6.4, unless they are allowed to be neglected by Section 406.2.5.

406.6.1.2 Redistribution of moments calculated by an elastic first-order analysis shall be permitted in accordance with Section 406.6.5.

406.6.2 Modeling of Members and Structural Systems

406.6.2.1 Floor or roof level moments shall be resisted by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffnesses and considering conditions of restraint.

406.6.2.2 For frames or continuous construction, consideration shall be given to the effect of floor and roof load patterns on transfer of moment to exterior and interior columns, and of eccentric loading due to other causes.

406.6.2.3 It shall be permitted to simplify the analysis model by the assumptions of (a), (b), or both:

- Solid slabs or one-way joist systems built integrally with supports, with clear spans not more than 3m., shall be permitted to be analyzed as continuous members on knife edge supports with spans equal to the clear spans of the member and width of support beams otherwise neglected;
- For frames or continuous construction, it shall be permitted to assume the intersecting member regions are rigid.

406.6.3 Section Properties

406.6.3.1 Factored Load Analysis

406.6.3.1.1 Floor Moment of inertia and cross-sectional area of members shall be calculated in accordance with Tables 406.6.3.1.1(a) or 406.6.3.1.1(b), unless a more rigorous analysis is used. If sustained lateral loads are present, I for columns and walls shall be divided by $(1 + \beta_{ds})$ where β_{ds} is the ratio of maximum factored sustained shear within a storey to the maximum factored shear in that storey associated with the same load combination.

Table 406.6.3.1.1(a) Moment of Inertia and Cross-Sectional Area Permitted for Elastic Analysis at Factored Load Level

Member and condition		Moment of Inertia	Cross-Sectional Area
Columns		$0.70I_g$	$1.0A_g$
Walls	Uncracked	$0.70I_g$	
	Cracked	$0.35I_g$	
Beams		$0.35I_g$	
Flat plates and flat slabs		$0.25I_g$	

Table 406.6.3.1.1(b) Alternative Moments of Inertia for Elastic Analysis at Factored Load

Member	Alternative Value of I for Elastic Analysis		
	Minimum	I	Maximum
Columns and walls	$0.35I_g$	$\left(0.80 + 25 \frac{A_{st}}{A_g}\right) \left(1 - \frac{M_u}{P_u h} - 0.5 \frac{P_u}{P_o}\right) I_g$	$0.875I_g$
Beams, flat plates, and flat slabs	$0.25I_g$	$(0.10 + 25\rho) \left(1.2 - 0.2 \frac{b_w}{d}\right) I_g$	$0.5I_g$

Note: For continuous flexural members, I shall be permitted to be taken as the average of values obtained for the critical positive and negative moment sections. P_u and M_u shall be calculated from the load combination under consideration, or the combination of P_u and M_u that produces the least value of I .

406.6.3.1.2 For factored lateral load analysis, it shall be permitted to assume $I = 0.5I_g$ for all members or to calculate I by a more detailed analysis, considering the reduced stiffness of all members under the loading conditions.

406.6.3.1.3 For factored lateral load analysis of two-way slab systems without beams, which are designated as part of the seismic-force-resisting system, I for slab members shall be defined by a model that is in substantial

agreement with results of comprehensive tests and analysis and I of other frame members shall be in accordance with Sections 406.6.3.1.1 and 406.6.3.1.2.

406.6.3.2 Service Load Analysis

406.6.3.2.1 Immediate and time-dependent deflections due to gravity loads shall be calculated in accordance with Section 424.2.

406.6.3.2.2 It shall be permitted to calculate immediate lateral deflections using a moment of inertia of 1.4 times I defined in Section 406.6.3.1, or using a more detailed analysis, but the value shall not exceed I_g .

406.6.4 Slenderness Effects, Moment Magnification Method

406.6.4.1 Unless Section 406.2.5 is satisfied, columns and stories in structures shall be designated as being nonsway or sway. Analysis of columns in nonsway frames or storeys shall be in accordance with Section 406.6.4.5. Analysis of columns in sway frames or storeys shall be in accordance with Section 406.6.4.6.

406.6.4.2 The cross-sectional dimensions of each member used in an analysis shall be within 10 percent of the specified member dimensions in construction documents or the analysis shall be repeated. If the stiffnesses of Table 406.6.3.1.1(b) are used in an analysis, the assumed member reinforcement ratio shall also be within 10 percent of the specified member reinforcement in construction documents.

406.6.4.3 It shall be permitted to analyze columns and stories in structures as nonsway frames if (a) or (b) is satisfied:

- The increase in column end moments due to second order effects does not exceed 5 percent of the first order end moments;
- Q in accordance with Section 406.6.4.4.1 does not exceed 0.05.

406.6.4.4 Stability Properties

406.6.4.4.1 The stability index for a storey, Q , shall be calculated by:

$$Q = \frac{\sum P_u \Delta_o}{V_{us} \ell_c} \quad (406.6.4.4.1)$$

where $\sum P_u$ and V_{us} are the total factored vertical load and horizontal storey shear, respectively, in the story

being evaluated, and Δ_o is the first-order relative lateral deflection between the top and the bottom of that storey due to V_{us} .

406.6.4.4.2 The critical buckling load, P_c , shall be calculated by:

$$P_c = \frac{\pi^2(EI)_{eff}}{(k\ell_u)^2} \quad (406.6.4.4.2)$$

406.6.4.4.3 The effective length factor k shall be calculated using E_c in accordance with Section 419.2.2 and I in accordance with Section 406.6.3.1.1. For nonsway members, k shall be permitted to be taken as 1.0, and for sway members, k shall be at least 1.0.

406.6.4.4.4 For non-composite columns, $(EI)_{eff}$ shall be calculated in accordance with (a), (b), or (c):

$$a. \quad (EI)_{eff} = \frac{0.4E_cI_g}{1 + \beta_{dns}} \quad (406.6.4.4.4a)$$

$$b. \quad (EI)_{eff} = \frac{0.2E_cI_g + E_sI_{se}}{1 + \beta_{dns}} \quad (406.6.4.4.4b)$$

$$c. \quad (EI)_{eff} = \frac{E_cI}{1 + \beta_{dns}} \quad (406.6.4.4.4c)$$

where β_{dns} shall be the ratio of maximum factored sustained axial load to maximum factored axial load associated with the same load combination and I in Eq. 406.6.4.4.4c is calculated according to Table 406.6.3.1.1(b) for columns and walls.

406.6.4.4.5 For composite columns, $(EI)_{eff}$ shall be calculated by Eq. 406.6.4.4.4b, Eq. 406.6.4.4.5, or from a more detailed analysis.

$$(EI)_{eff} = \frac{(0.2E_cI_g)}{1 + \beta_{dns}} + E_sI_{sx} \quad (406.6.4.4.5)$$

406.6.4.5 Moment Magnification Method: Nonsway Frames

406.6.4.5.1 The factored moment used for design of columns and walls, M_c , shall be the first-order factored moment M_2 amplified for the effects of member curvature.

$$M_c = \delta M_2 \quad (406.6.4.5.1)$$

406.6.4.5.2 Magnification factor δ shall be calculated by:

$$\delta = \frac{C_m}{1 - \frac{P_u}{0.75P_c}} \geq 1.0 \quad (406.6.4.5.2)$$

406.6.4.5.3 C_m shall be in accordance with (a) or (b):

a. For columns without transverse loads applied between supports:

$$C_m = 0.6 - 0.4 \frac{M_1}{M_2} \quad (406.6.4.5.3a)$$

where M_1/M_2 is negative if the column is bent in single curvature, and positive if bent in double curvature;

b. For columns with transverse loads applied between supports.

$$C_m = 1.0 \quad (406.6.4.5.3b)$$

406.6.4.5.4 M_2 in Eq. 406.6.4.5.1 shall be at least $M_{2,min}$ calculated according to Eq. 406.6.4.5.4 about each axis separately.

$$M_{2,min} = P_u(15 + 0.03h) \quad (406.6.4.5.4)$$

If $M_{2,min}$ exceeds M_2 , C_m shall be taken equal to 1.0 or calculated based on the ratio of the calculated end moments M_1/M_2 , using Eq. 406.6.4.5.3a.

406.6.4.6 Moment Magnification Method: Sway Frames

406.6.4.6.1 Moments M_1 and M_2 at the ends of an individual column shall be calculated by (a) and (b).

$$a. \quad M_1 = M_{1ns} + \delta_s M_{1s} \quad (406.6.4.6.1a)$$

$$b. \quad M_2 = M_{2ns} + \delta_s M_{2s} \quad (406.6.4.6.1b)$$

406.6.4.6.2 The moment magnifier δ_s shall be calculated by (a), (b), or (c). If δ_s exceeds 1.5, only (b) or (c) shall be permitted:

$$a. \quad \delta_s = \frac{1}{1 - Q} \geq 1 \quad (406.6.4.6.2a)$$

$$b. \quad \delta_s = \frac{1}{1 - \frac{\sum P_u}{0.75 \sum P_c}} \geq 1 \quad (406.6.4.6.2b)$$

c. Second-order elastic analysis.

where $\sum P_u$ is the summation of all the factored vertical loads in a storey and $\sum P_c$ is the summation for all sway resisting columns in a storey. P_c is calculated using Eq. 406.6.4.4.2 with k determined for sway members from Section 406.6.4.4.3 and $(EI)_{eff}$ from Section 406.6.4.4.4 or 406.6.4.4.5 as appropriate with β_{ds} substituted for β_{ans} .

406.6.4.6.3 Flexural members shall be designed for the total magnified end moments of the columns at the joint.

406.6.4.6.4 Second-order effects shall be considered along the length of columns in sway frames. It shall be permitted to account for these effects using Section 406.6.4.5, where C_m is calculated using M_1 and M_2 from Section 406.6.4.6.1.

406.6.5 Redistribution of Moments in Continuous Flexural Members

406.6.5.1 Except where approximate values for moments are used in accordance with Section 406.5, where moments have been calculated in accordance with Section 406.8, or where moments in two-way slabs are determined using pattern loading specified in Section 406.4.3.3, reduction of moments at sections of maximum negative or maximum positive moment calculated by elastic theory shall be permitted for any assumed loading arrangement if (a) and (b) are satisfied:

- a. Flexural members are continuous;
- b. $\epsilon_t \geq 0.0075$ at the section at which moment is reduced.

406.6.5.2 For prestressed members, moments include those due to factored loads and those due to reactions induced by prestressing.

406.6.5.3 At the section where the moment is reduced, redistribution shall not exceed the lesser of $1000\epsilon_t$ percent and 20 percent.

406.6.5.4 The reduced moment shall be used to calculate redistributed moments at all other sections within the spans such that static equilibrium is maintained after redistribution of moments for each loading arrangement.

406.6.5.5 Shears and support reactions shall be calculated in accordance with static equilibrium considering the redistributed moments for each loading arrangement.

406.7 Elastic Second-order Analysis

406.7.1 General

406.7.1.1 An elastic second-order analysis shall consider the influence of axial loads, presence of cracked regions along the length of the member, and effects of load duration. These considerations are satisfied using the cross-sectional properties defined in Section 406.7.2.

406.7.1.2 Slenderness effects along the length of a column shall be considered. It shall be permitted to calculate these effects using Section 406.6.4.5.

406.7.1.3 The cross-sectional dimensions of each member used in an analysis to calculate slenderness effects shall be within 10 percent of the specified member dimensions in construction documents or the analysis shall be repeated.

406.7.1.4 Redistribution of moments calculated by an elastic second-order analysis shall be permitted in accordance with Section 406.6.5.

406.7.2 Section Properties

406.7.2.1 Factored Load Analysis

406.7.2.1.1 It shall be permitted to use section properties calculated in accordance with Section 406.6.3.1.

406.7.2.2 Service Load Analysis

406.7.2.2.1 Immediate and time-dependent deflections due to gravity loads shall be calculated in accordance with Section 424.2.

406.7.2.2.2 Alternatively, it shall be permitted to calculate immediate deflections using a moment of inertia of 1.4 times I given in Section 406.6.3.1, or calculated using a more detailed analysis, but the value shall not exceed I_g .

406.8 Inelastic Second-Order Analysis

406.8.1 General

406.8.1.1 An inelastic second-order analysis shall consider material nonlinearity, member curvature and lateral drift, duration of loads, shrinkage and creep, and interaction with the supporting foundation.

406.8.1.2 An inelastic second-order analysis procedure shall have been shown to result in prediction of strength in substantial agreement with results of comprehensive

tests of statically indeterminate reinforced concrete structures.

406.8.1.3 Slenderness effects along the length of a column shall be considered. It shall be permitted to calculate these effects using Section 406.6.4.5.

406.8.1.4 The cross-sectional dimensions of each member used in an analysis to calculate slenderness effects shall be within 10 percent of the specified member dimensions in construction documents or the analysis shall be repeated.

406.8.1.5 Redistribution of moments calculated by an inelastic second-order analysis shall not be permitted.

406.9 Acceptability of Finite Element Analysis

406.9.1 Finite element analysis to determine load effects shall be permitted.

406.9.2 The finite element model shall be appropriate for its intended purpose.

406.9.3 For inelastic analysis, a separate analysis shall be performed for each factored load combination.

406.9.4 The licensed design professional shall confirm that the results are appropriate for the purposes of the analysis.

406.9.5 The cross-sectional dimensions of each member used in an analysis shall be within 10 percent of the specified member dimensions in construction documents or the analysis shall be repeated.

406.9.6 Redistribution of moments calculated by an inelastic analysis shall not be permitted.

SECTION 407 ONE-WAY SLABS

407.1 Scope

407.1.1 This section shall apply to the design of non-prestressed and prestressed slabs reinforced for flexure in one direction, including:

- a. Solid slabs;
- b. Slabs cast on stay-in-place, non-composite steel deck;
- c. Composite slabs of concrete elements constructed in separate placements but connected so that all elements resist loads as a unit;
- d. Precast, prestressed hollow-core slabs.

407.2 General

407.2.1 The effects of concentrated loads and openings shall be considered in design.

407.2.2 Materials

407.2.2.1 Design properties for concrete shall be selected to be in accordance with Section 419.

407.2.2.2 Design properties for steel reinforcement shall be selected to be in accordance with Section 420.

407.2.2.3 Materials, design and detailing requirements for embedments in concrete shall be in accordance with Section 420.7.

407.2.3 Connection to Other Members

407.2.3.1 For cast-in-place construction, slab-column joints shall satisfy Section 415.

407.2.3.2 For precast construction, connections shall satisfy the force transfer requirements of Section 416.2.

407.3 Design Limits

407.3.1 Minimum Slab Thickness

407.3.1.1 For solid non-prestressed slabs not supporting or attached to partitions or other construction likely to be damaged by large deflections, overall slab thickness h shall not be less than the limits in Table 407.3.1.1, unless the calculated deflection limits of Section 407.3.2 are satisfied.

Table 407.3.1.1 Minimum Thickness of Solid Non-Prestressed One-Way Slabs

Support condition	Minimum h ^[1]
Simply supported	$\ell/20$
One end continuous	$\ell/24$
Both ends continuous	$\ell/28$
Cantilever	$\ell/10$

^[1] Expression applicable for normal weight concrete and $f_y = 420$ MPa. For other cases, minimum h shall be modified in accordance with Sections 407.3.1.1.1 through 407.3.1.1.3, as appropriate.

407.3.1.1.1 For f_y other than 420 MPa, the expressions in Table 407.3.1.1 shall be multiplied by $(0.4 + f_y/700)$.

407.3.1.1.2 For non-prestressed slabs made of lightweight concrete having w_c in the range of 1440 to 1840 kg/m³, the expressions in Table 407.3.1.1 shall be multiplied by the greater of (a) and (b):

- a. $1.65 - 0.0003w_c$
- b. 1.09

407.3.1.1.3 For non-prestressed composite slabs made of a combination of lightweight and normal weight concrete that are shored during construction, where the lightweight concrete is in compression, the modifier of Section 407.3.1.1.2 shall apply.

407.3.1.2 The thickness of a concrete floor finish shall be permitted to be included in h if it is placed monolithically with the floor slab, or if the floor finish is designed to be composite with the floor slab in accordance with Section 416.4.

407.3.2 Calculated Deflection Limits

407.3.2.1 For non-prestressed slabs not satisfying Section 407.3.1 and for prestressed slabs, immediate and time-dependent deflections shall be calculated in accordance with Section 424.2 and shall not exceed the limits in Section 424.2.2.

407.3.2.2 For non-prestressed composite concrete slabs satisfying Section 407.3.1, deflections occurring after the member becomes composite need not be calculated. Deflections occurring before the member becomes composite shall be investigated, unless the pre-composite thickness also satisfies Section 407.3.1.

407.3.3 Reinforcement Strain Limit in Non-Prestressed Slabs

407.3.3.1 For non-prestressed slabs, ϵ_t shall be at least 0.004.

407.3.4 Stress Limits in Prestressed Slabs

407.3.4.1 Prestressed slabs shall be classified as Class U, T, or C in accordance with Section 424.5.2.

407.3.4.2 Stresses in prestressed slabs immediately after transfer and at service loads shall not exceed the permissible stresses in Sections 424.5.3 and 424.5.4.

407.4 Required Strength

407.4.1 General

407.4.1.1 Required strength shall be calculated in accordance with the factored load combinations in Section 405.

407.4.1.2 Required strength shall be calculated in accordance with the analysis procedures in Section 406.

407.4.1.3 For prestressed slabs, effects of reactions induced by prestressing shall be considered in accordance with Section 405.3.11.

407.4.2 Factored Moment

407.4.2.1 For slabs built integrally with supports, M_u at the support shall be permitted to be calculated at the face of support.

407.4.3 Factored Shear

407.4.3.1 For slabs built integrally with supports, V_u at the support shall be permitted to be calculated at the face of support.

407.4.3.2 Sections between the face of support and a critical section located d from the face of support for non-prestressed slabs or $h/2$ from the face of support for prestressed slabs shall be permitted to be designed for V_u at that critical section if (a) through (c) are satisfied:

- a. Support reaction, in direction of applied shear, introduces compression into the end region of the slab;
- b. Loads are applied at or near the top surface of the slab;

- c. No concentrated load occurs between the face of support and critical section.

407.5 Design Strength

407.5.1 General

407.5.1.1 For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$ including (a) and (b):

a. $\phi M_n \geq M_u$

b. $\phi V_n \geq V_u$

Interaction between load effects shall be considered.

407.5.1.2 ϕ shall be determined in accordance with Section 421.2.

407.5.2 Moment

407.5.2.1 M_n shall be calculated in accordance with Section 422.3.

407.5.2.2 For prestressed slabs, external tendons shall be considered as unbonded tendons in calculating flexural strength, unless the external tendons are effectively bonded to the concrete section along its entire length.

407.5.2.3 If primary flexural reinforcement in a slab that is considered to be a T-beam flange is parallel to the longitudinal axis of the beam, reinforcement perpendicular to the longitudinal axis of the beam shall be provided in the top of the slab in accordance with (a) and (b). This provision does not apply to joist construction:

- Slab reinforcement perpendicular to the beam shall be designed to resist the factored load on the overhanging slab width assumed to act as a cantilever;
- Only the effective overhanging slab width in accordance with Section 406.3.2 need be considered.

407.5.3 Shear

407.5.3.1 V_n shall be calculated in accordance with Section 422.5.

407.5.3.2 For composite concrete slabs, horizontal shear strength, V_{nh} , shall be calculated in accordance with Section 416.4.

407.6 Reinforcement Limits

407.6.1 Minimum Flexural Reinforcement in Non-Prestressed Slabs

407.6.1.1 A minimum area of flexural reinforcement $A_{s,min}$ shall be provided in accordance with Table 407.6.1.1.

Table 407.6.1.1 $A_{s,min}$ for Non-Prestressed One-Way Slabs

Reinforcement Type	f_y , MPa	$A_{s,min}$	
Deformed bars	<420	$0.0020A_g$	
Deformed bars or welded wire reinforcement	≥ 420	Greater of:	$\frac{0.0018 \times 420}{F_y} A_g$
			$0.0014A_g$

407.6.2 Minimum Flexural Reinforcement in Prestressed Slabs

407.6.2.1 For slabs with bonded prestressed reinforcement, total quantity of A_s and A_{ps} shall be adequate to develop a factored load at least 1.2 times the cracking load calculated on the basis of f_r as given in Section 419.2.3.

407.6.2.2 For slabs with both flexural and shear design strength at least twice the required strength, Section 407.6.2.1 need not be satisfied.

407.6.2.3 For slabs with unbonded tendons, the minimum area of bonded deformed longitudinal reinforcement, $A_{s,min}$, shall be:

$$A_{s,min} \geq 0.004A_{ct} \quad (407.6.2.3)$$

where A_{ct} is the area of that part of the cross section between the flexural tension face and the centroid of the gross section.

407.6.3 Minimum Shear Reinforcement

407.6.3.1 A minimum area of shear reinforcement, $A_{v,min}$ shall be provided in all regions where $V_u > \phi V_c$. For precast prestressed hollow-core slabs with untopped $h > 315\text{mm}$, $A_{v,min}$ shall be provided in all regions where $V_u > 0.5\phi V_{cw}$.

407.6.3.2 If shown by testing that the required M_n and V_n can be developed, Section 407.6.3.1 need not be satisfied. Such tests shall simulate effects of differential settlement, creep, shrinkage, and temperature change, based on a realistic assessment of these effects occurring in service.

407.6.3.3 If shear reinforcement is required, $A_{v,min}$, shall be in accordance with Section 409.6.3.3.

407.6.4 Minimum Shrinkage and Temperature Reinforcement

407.6.4.1 Reinforcement shall be provided to resist shrinkage and temperature stresses in accordance with Section 424.4.

407.6.4.2 If prestressed shrinkage and temperature reinforcement in accordance with Section 424.4.4 is used, Sections 407.6.4.2.1 through 407.6.4.2.3 shall apply.

407.6.4.2.1 For monolithic, cast-in-place, post-tensioned beam-and-slab construction, gross concrete area shall consist of the total beam area including the slab thickness and the slab area within half the clear distance to adjacent beam webs. It shall be permitted to include the effective force in beam tendons in the calculation of total prestress force acting on gross concrete area.

407.6.4.2.2 If slabs are supported on walls or not cast monolithically with beams, gross concrete area is the slab section tributary to the tendon or tendon group.

407.6.4.2.3 At least one tendon is required in the slab between faces of adjacent beams or walls.

407.7 Reinforcement Detailing

407.7.1 General

407.7.1.1 Concrete cover for reinforcement shall be in accordance with Section 420.6.1.

407.7.1.2 Development lengths of deformed and prestressed reinforcement shall be in accordance with Section 425.4.

407.7.1.3 Splices of deformed reinforcement shall be in accordance with Section 425.5.

407.7.1.4 Bundled bars shall be in accordance with Section 425.6.

407.7.2 Reinforcement Spacing

407.7.2.1 Minimum spacing s shall be in accordance with Section 425.2.

407.7.2.2 For non-prestressed and Class C prestressed slabs, spacing of bonded longitudinal reinforcement closest to the tension face shall not exceed s calculated in accordance with Section 424.3.

407.7.2.3 Maximum spacing s of deformed reinforcement shall be the lesser of $3h$ and 450 mm.

407.7.2.4 Spacing of reinforcement required by Section 407.5.2.3 shall not exceed the lesser of $5h$ and 450 mm.

407.7.3 Flexural Reinforcement in Non-Prestressed Slabs

407.7.3.1 Calculated tensile or compressive force in reinforcement at each section of the slab shall be developed on each side of that section.

407.7.3.2 Critical locations for development of reinforcement are points of maximum stress and points along the span where bent or terminated tension reinforcement is no longer required to resist flexure.

407.7.3.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance at least the greater of d and $12d_b$, except at supports of simply-supported spans and at free ends of cantilevers.

407.7.3.4 Continuing flexural tension reinforcement shall have an embedment length at least ℓ_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

407.7.3.5 Flexural tension reinforcement shall not be terminated in a tension zone unless (a), (b), or (c) is satisfied:

- $V_u \leq (2/3)\phi V_n$ at the cutoff point;
- For 36 mm ϕ bars and smaller, continuing reinforcement provides double the area required for flexure at the cutoff point and $V_u \leq (3/4)\phi V_n$;
- Stirrup area in excess of that required for shear is provided along each terminated bar or wire over a distance $3/4d$ from the termination point. Excess stirrup area shall be not less than $0.41b_w s/f_{yt}$. Spacing s shall not exceed $d/(8\beta_b)$.

407.7.3.6 Adequate anchorage shall be provided for tension reinforcement where reinforcement stress is not directly proportional to moment, such as in sloped, stepped, or tapered slabs, or where tension reinforcement is not parallel to the compression face.

407.7.3.7 In slabs with spans not exceeding 3 m, welded wire reinforcement, with wire size not exceeding MW30 or MD30, shall be permitted to be curved from a point near the top of slab over the support to a point near the bottom of slab at mid-span, provided such reinforcement is continuous over, or developed at, the support.

407.7.3.8 Termination of Reinforcement

407.7.3.8.1 At simple supports, at least one-third of the maximum positive moment reinforcement shall extend along the slab bottom into the support. For precast slabs, such reinforcement shall extend at least to the center of the bearing length.

407.7.3.8.2 At other supports, at least one-fourth of the maximum positive moment reinforcement shall extend along the slab bottom into the support at least 150 mm.

407.7.3.8.3 At simple supports and points of inflection, d_b for positive moment tension reinforcement shall be limited such that ℓ_d for that reinforcement satisfies (a) or (b). If reinforcement terminates beyond the centerline of supports by a standard hook or a mechanical anchorage at least equivalent to a standard hook, (a) or (b) need not be satisfied.

- a. $\ell_d \leq (1.3M_n/V_u + \ell_a)$ if end of reinforcement is confined by a compressive reaction;
- b. $\ell_d \leq (M_n/V_u + \ell_a)$ if end of reinforcement is not confined by a compressive reaction.

where M_n is calculated assuming all reinforcement at the section is stressed to f_y and V_u is calculated at the section.

At a support, ℓ_a is the embedment length beyond the center of the support.

At a point of inflection, ℓ_a is the embedment length beyond the point of inflection, limited to the greater of d and $12d_b$.

407.7.3.8.4 At least one-third of the negative moment reinforcement at a support shall have an embedment length beyond the point of inflection at least the greatest of d , $12d_b$, and $\ell_n/16$.

407.7.4 Flexural Reinforcement in Prestressed Slabs

407.7.4.1 External tendons shall be attached to the member in a manner that maintains the specified eccentricity between the tendons and the concrete centroid through the full range of anticipated member deflections.

407.7.4.2 If non-prestressed reinforcement is required to satisfy flexural strength, the detailing requirements of Section 407.7.3 shall be satisfied.

407.7.4.3 Termination of Prestressed Reinforcement

407.7.4.3.1 Post-tensioned anchorage zones shall be designed and detailed in accordance with Section 425.9.

407.7.4.3.2 Post-tensioning anchorages and couplers shall be designed and detailed in accordance with Section 425.8.

407.7.4.4 Termination of Deformed Reinforcement in Slabs with Unbonded Tendons

407.7.4.4.1 Length of deformed reinforcement required by Section 407.6.2.3 shall be in accordance with (a) and (b):

- a. At least $\ell_n/3$ in positive moment areas and be centered in those areas;
- b. At least $\ell_n/6$ on each side of the face of support.

407.7.5 Shear Reinforcement

407.7.5.1 If shear reinforcement is required, transverse reinforcement shall be detailed according to Section 409.7.6.2.

407.7.6 Shrinkage and Temperature Reinforcement

407.7.6.1 Shrinkage and temperature reinforcement in accordance with Section 407.6.4 shall be placed perpendicular to flexural reinforcement.

407.7.6.2 Non-Prestressed Reinforcement

407.7.6.2.1 Spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of $5h$ and 450 mm.

407.7.6.3 Prestressed Reinforcement

407.7.6.3.1 Spacing of slab tendons required by Section 407.6.4.2, and the distance between face of beam or wall to the nearest slab tendon, shall not exceed 1.8 m.

407.7.6.3.2 If spacing of slab tendons exceeds 1.4 m., additional deformed shrinkage and temperature reinforcement conforming to Section 424.4.3 shall be provided parallel to the tendons, except Section 424.4.3.4 need not be satisfied.

In calculating the area of additional reinforcement, it shall be permitted to take the gross concrete area in Table 424.4.3.2 as the slab area between faces of beams. This shrinkage and temperature reinforcement shall extend from the slab edge for a distance not less than the slab tendon spacing.

SECTION 408

TWO-WAY SLABS

408.1 Scope

408.1.1 This section shall apply to the design of non-prestressed and prestressed slabs reinforced for flexure in two directions, with or without beams between supports, including (a) through (d):

- a. Solid slabs;
- b. Slabs cast on stay-in-place, non-composite steel deck;
- c. Composite slabs of concrete elements constructed in separate placements but connected so that all elements resist loads as a unit;
- d. Two-way joist systems in accordance with Section 408.8.

408.2 General

408.2.1 A slab system shall be permitted to be designed by any procedure satisfying equilibrium and geometric compatibility, provided that design strength at every section is at least equal to required strength, and all serviceability requirements are satisfied. The direct design method of Section 408.10 or the equivalent frame method of Section 408.11 is permitted for design where applicable.

408.2.2 The effects of concentrated loads and openings shall be considered in design.

408.2.3 Slabs prestressed with an average effective compressive stress less than 0.9 MPa shall be designed as non-prestressed slabs.

408.2.4 A drop panel in a non-prestressed slab, where used to reduce the minimum required thickness in accordance with Section 408.3.1.1 or the quantity of deformed negative moment reinforcement at a support in accordance with Section 408.5.2.2, shall satisfy (a) and (b):

- a. The drop panel shall project below the slab at least 1/4 of the adjacent slab thickness;
- b. The drop panel shall extend in each direction from the centerline of support a distance not less than 1/6 the span length measured from center-to-center of supports in that direction.

408.2.5 A shear cap, where used to increase the critical section for shear at a slab-column joint, shall project below the slab soffit and extend horizontally from the face of the column a distance at least equal to the thickness of the projection below the slab soffit.

408.2.6 Materials

408.2.6.1 Design properties for concrete shall be selected to be in accordance with Section 419.

408.2.6.2 Design properties for steel reinforcement shall be selected to be in accordance with Section 420.

408.2.6.3 Materials, design and detailing requirements for embedments in concrete shall be in accordance with Section 420.7.

408.2.7 Connections to Other Members

408.2.7.1 Connections of two-way slabs to supporting members shall be in accordance with Section 415.

408.3 Design Limits

408.3.1 Minimum Slab Thickness

408.3.1.1 For non-prestressed slabs without interior beams spanning between supports on all sides, having a maximum ratio of long-to-short span of 2, overall slab thickness h shall not be less than the limits in Table 408.3.1.1, and shall be at least the value in (a) or (b), unless the calculated deflection limits of Section 408.3.2 are satisfied:

- a. Slabs without drop panels as given in Section 408.2.4 **125 mm.**
- b. Slabs drop panels as given in Section 408.2.4 **100 mm.**

Table 408.3.1.1 Minimum Thickness of Non-Prestressed Two-Way Slabs without Interior Beams (mm.)^[1]

f_y , MPa ^[2]	Without drop panels ^[3]		With drop panels ^[3]			
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams ^[4]		Without edge beams	With edge beams ^[4]	
280	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
420	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
520	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$	$\ell_n/31$	$\ell_n/34$	$\ell_n/34$

^[1] ℓ_n is the clear span in the long direction, measured face-to-face of supports (mm.).

^[2] For f_y between the values given in the table, minimum thickness shall be calculated by linear interpolation

^[3] Drop panels as given in Section 408.2.4.

^[4] Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall be calculated in accordance with Section 408.10.2.7. Exterior panels shall be considered to be without edge beams if α_f is less than 0.8.

408.3.1.2 For non-prestressed slabs with beams spanning between supports on all sides, overall slab thickness h shall satisfy the limits in Table 408.3.1.2, unless the calculated deflection limits of Section 408.3.2 are satisfied.

Table 408.3.1.2 Minimum Thickness of Non-Prestressed Two-Way Slabs with Beams Spanning between Supports on All Sides

$\alpha_{fm}^{[1]}$	Minimum h , mm		
$\alpha_{fm} \leq 0.2$	Section 408.3.1.1 applies		(a)
$0.22 < \alpha_{fm} \leq 2.0$	Greater of:	$\frac{\ell_n \left(0.8 \frac{f_y}{1,400} \right)}{36 + 5\beta(\alpha_{fm} - 0.2)}$	(b) ^{[2][3]}
		125	(c)
$\alpha_{fm} > 2.0$	Greater of:	$\frac{\ell_n \left(0.8 \frac{f_y}{1,400} \right)}{36 + 9\beta}$	(d) ^{[2][3]}
		90	(e)

^[1] α_{fm} is the average value of α_f for all beams on edges of a panel and α_f shall be calculated in accordance with Section 408.10.2.7.

^[2] ℓ_n is the clear span in the long direction, measured face-to-face of beams (mm.).

^[3] β is the ratio of clear spans in long to short directions of slab.

408.3.1.2.1 At discontinuous edges of slabs conforming to Section 408.3.1.2, an edge beam with $\alpha_f \geq 0.80$ shall be provided, or the minimum thickness required by (b) or (d) of Table 408.3.1.2 shall be increased by at least 10 percent in the panel with a discontinuous edge.

408.3.1.3 The thickness of a concrete floor finish shall be permitted to be included in h if it is placed

monolithically with the floor slab, or if the floor finish is designed to be composite with the floor slab in accordance with Section 416.4.

408.3.1.4 If single- or multiple-leg stirrups are used as shear reinforcement, the slab thickness shall be sufficient to satisfy the requirements for d in Section 422.6.7.1.

408.3.2 Calculated Deflection Limits

408.3.2.1 Immediate and time-dependent deflections shall be calculated in accordance with Section 424.2 and shall not exceed the limits in Section 424.2.2 for two-way slabs given in (a) through (c):

- a. Non-prestressed slabs not satisfying Section 408.3.1;
- b. Non-prestressed slabs without interior beams spanning between the supports on all sides and having a ratio of long-to-short span exceeding 2.0;
- c. Prestressed slabs.

408.3.2.2 For non-prestressed composite concrete slabs satisfying Sections 408.3.1.1 or 408.3.1.2, deflections occurring after the member becomes composite need not be calculated. Deflections occurring before the member becomes composite shall be investigated, unless the pre-composite thickness also satisfies Sections 408.3.1.1 or 408.3.1.2.

408.3.3 Reinforcement Strain Limit in Non-Prestressed Slabs

408.3.3.1 For non-prestressed slabs, ϵ_t shall be at least 0.004.

408.3.4 Stress Limits in Prestressed Slabs

408.3.4.1 Prestressed slabs shall be designed as Class U with $f_t \leq 0.50\sqrt{f'_c}$. Other stresses in prestressed slabs immediately after transfer and at service loads shall not exceed the permissible stresses in Sections 424.5.3 and 424.5.4.

408.4 Required Strength

408.4.1 General

408.4.1.1 Required strength shall be calculated in accordance with the factored load combinations in Section 405.

408.4.1.2 Required strength shall be calculated in accordance with the analysis procedures given in Section

406. Alternatively, the provisions of Section 408.10 for the direct design method shall be permitted for the analysis of non-prestressed slabs and the provisions of Section 408.11 for the equivalent frame method shall be permitted for the analysis of non-prestressed and prestressed slabs, except Sections 408.11.6.5 and 408.11.6.6 shall not apply to prestressed slabs.

408.4.1.3 For prestressed slabs, effects of reactions induced by prestressing shall be considered in accordance with Section 405.3.11.

408.4.1.4 For a slab system supported by columns or walls, dimensions c_1 , c_2 , and ℓ_n shall be based on an effective support area. The effective support area is the intersection of the bottom surface of the slab, or drop panel or shear cap if present, with the largest right circular cone, right pyramid, or tapered wedge whose surfaces are located within the column and the capital or bracket and are oriented no greater than 45 degrees to the axis of the column.

408.4.1.5 A column strip is a design strip with a width on each side of a column centerline equal to the lesser of $0.25\ell_2$ and $0.25\ell_1$. A column strip shall include beams within the strip, if present.

408.4.1.6 A middle strip is a design strip bounded by two column strips.

408.4.1.7 A panel is bounded by column, beam, or wall centerlines on all sides.

408.4.1.8 For monolithic or fully composite construction supporting two-way slabs, a beam includes that portion of slab, on each side of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness.

408.4.1.9 Combining the results of a gravity load analysis with the results of a lateral load analysis shall be permitted.

408.4.2 Factored Moment

408.4.2.1 For slabs built integrally with supports, M_u at the support shall be permitted to be calculated at the face of support, except if analyzed in accordance with Section 408.4.2.2.

408.4.2.2 For slabs analyzed using the direct design method or the equivalent frame method, M_u at the support shall be located in accordance with Section 408.10 or 408.11, respectively.

408.4.2.3 Factored Slab Moment Resisted by the Column

408.4.2.3.1 If gravity load, wind, earthquake, or other effects cause a transfer of moment between the slab and column, a fraction of M_{sc} , the factored slab moment resisted by the column at a joint, shall be transferred by flexure in accordance with Sections 408.4.2.3.2 through 408.4.2.3.5.

408.4.2.3.2 The fraction of factored slab moment resisted by the column, $\gamma_f M_{sc}$, shall be assumed to be transferred by flexure, where γ_f shall be calculated by:

$$\gamma_f = \frac{1}{1 + \left(\frac{2}{3}\right) \sqrt{\frac{b_1}{b_2}}} \quad (408.4.2.3.2)$$

408.4.2.3.3 The effective slab width b_{slab} for resisting $\gamma_f M_{sc}$ shall be the width of column or capital plus **1.5h** of slab or drop panel on either side of column or capital.

408.4.2.3.4 For non-prestressed slabs, where the limitations on v_{ug} and ϵ_t in Table 408.4.2.3.4 are satisfied, γ_f shall be permitted to be increased to the maximum modified values provided in Table 408.4.2.3.4, where v_c is calculated in accordance with Section 422.6.5, and v_{ug} is the factored shear stress on the slab critical section for two-way action due to gravity loads without moment transfer.

408.4.2.3.5 Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist moment on the effective slab width defined in Sections 408.4.2.3.2 and 408.4.2.3.3.

Table 408.4.2.3.4 Maximum Modified Values of γ_f for Non-Prestressed Two-Way Slabs

Column Location	Span Direction	v_{ug}	ϵ_t (within b_{slab})	Maximum Modified γ_f
Corner column	Either direction	$\leq 0.5\phi v_c$	≥ 0.004	1.0
Edge column	Perpendicular to the edge	$\leq 0.75\phi v_c$	≥ 0.004	1.0
	Parallel to the edge	$\leq 0.4\phi v_c$	≥ 0.010	$\frac{1.25}{1 + \left(\frac{2}{3}\right) \sqrt{\frac{b_1}{b_2}}} \leq 1.0$
Interior column	Either direction	$\leq 0.4\phi v_c$	≥ 0.010	$\frac{1.25}{1 + \left(\frac{2}{3}\right) \sqrt{\frac{b_1}{b_2}}} \leq 1.0$

408.4.2.3.6 The fraction of M_{sc} not calculated to be resisted by flexure shall be assumed to be resisted by eccentricity of shear in accordance with Section 408.4.4.2.

408.4.3 Factored One-Way Shear

408.4.3.1 For slabs built integrally with supports, V_u at the support shall be permitted to be calculated at the face of support.

408.4.3.2 Sections between the face of support and a critical section located d from the face of support for non-prestressed slabs and $h/2$ from the face of support for prestressed slabs shall be permitted to be designed for V_u at that critical section if (a) through (c) are satisfied:

- Support reaction, in direction of applied shear, introduces compression into the end regions of the slab;
- Loads are applied at or near the top surface of the slab;
- No concentrated load occurs between the face of support and critical section.

408.4.4 Factored Two-Way Shear

408.4.4.1 Critical Section

408.4.4.1.1 Slabs shall be evaluated for two-way shear in the vicinity of columns, concentrated loads, and reaction areas at critical sections in accordance with Section 422.6.4.

408.4.4.1.2 Slabs reinforced with stirrups or headed shear stud reinforcement shall be evaluated for two-way shear at critical sections in accordance with Section 422.6.4.2.

408.4.4.1.3 Slabs reinforced with shearheads shall be evaluated for two-way shear at critical sections in accordance with Section 422.6.9.8.

408.4.4.2 Factored Two-Way Shear Stress Due to Shear and Factored Slab Moment Resisted by the Column

408.4.4.2.1 For two-way shear with factored slab moment resisted by the column, factored shear stress v_u shall be calculated at critical sections in accordance with Section 408.4.4.1. Factored shear stress v_u corresponds to a combination of v_{ug} and the shear stress produced by $\gamma_v M_{sc}$, where γ_v is given in Section 408.4.4.2.2 and M_{sc} is given in Section 408.4.2.3.1.

408.4.4.2.2 The fraction of M_{sc} transferred by eccentricity of shear, $\gamma_v M_{sc}$, shall be applied at the centroid of the critical section in accordance with Section 408.4.4.1, where:

$$\gamma_v = 1 - \gamma_f \quad (408.4.4.2.2)$$

408.4.4.2.3 The factored shear stress resulting from $\gamma_v M_{sc}$ shall be assumed to vary linearly about the centroid of the critical section in accordance with Section 408.4.4.1.

408.5 Design Strength

408.5.1 General

408.5.1.1 For each applicable factored load combination, design strength shall satisfy $\phi S_n \geq U$, including (a) through (d). Interaction between load effects shall be considered.

- $\phi M_n \geq M_u$ at all sections along the span in each direction;
- $\phi M_n \geq \gamma_f M_{sc}$ within b_{slab} as defined in Section 408.4.2.3.3;
- $\phi V_n \geq V_u$ at all sections along the span in each direction for one-way shear;
- $\phi v_n \geq v_u$ at the critical sections defined in Section 408.4.4.1 for two-way shear.

408.5.1.2 ϕ shall be in accordance with Section 421.2.

408.5.1.3 If shearheads are provided, Sections 422.6.9 and 408.5.1.1(a) shall be satisfied in the vicinity of the column. Beyond each arm of the shearhead, Section 408.5.1.1(a) through (d) shall apply.

408.5.2 Moment

408.5.2.1 M_n shall be calculated in accordance with Section 422.3.

408.5.2.2 In calculating M_n for non-prestressed slabs with a drop panel, the thickness of the drop panel below the slab shall not be assumed to be greater than 1/4 the distance from the edge of drop panel to the face of column or column capital.

408.5.2.3 In calculating M_n for prestressed slabs, external tendons shall be considered as unbonded unless the external tendons are effectively bonded to the slab along its entire length.

408.5.3 Shear

408.5.3.1 Design shear strength of slabs in the vicinity of columns, concentrated loads, or reaction areas shall be the more severe of Sections 408.5.3.1.1 and 408.5.3.1.2.

408.5.3.1.1 For one-way shear, where each critical section to be investigated extends in a plane across the entire slab width, V_n shall be calculated in accordance with Section 422.5.

408.5.3.1.2 For two-way shear, v_n shall be calculated in accordance with Section 422.6.

408.5.3.2 For composite concrete slabs, horizontal shear strength, V_{nh} , shall be calculated in accordance with Section 416.4.

408.5.4 Openings in Slab Systems

408.5.4.1 Openings of any size shall be permitted in slab systems if shown by analysis that all strength and serviceability requirements, including the limits on deflections, are satisfied.

408.5.4.2 As an alternative to Section 408.5.4.1, openings shall be permitted in slab systems without beams in accordance with (a) through (d):

- Openings of any size shall be permitted in the area common to intersecting middle strips, but the total quantity of reinforcement in the panel shall be at least that required for the panel without the opening;
- At two intersecting column strips, not more than 1/8 the width of column strip in either span shall be interrupted by openings. A quantity of reinforcement at least equal to that interrupted by an opening shall be added on the sides of the opening;
- At the intersection of one column strip and one middle strip, not more than 1/4 of the reinforcement in either strip shall be interrupted by openings. A quantity of reinforcement at least equal to that interrupted by an opening shall be added on the sides of the opening;
- If an opening is located within a column strip or closer than $10h$ from a concentrated load or reaction area, Section 422.6.4.3 for slabs without shearheads or Section 422.6.9.9 for slabs with shearheads shall be satisfied.

408.6 Reinforcement Limits**408.6.1 Minimum Flexural Reinforcement in Non-Prestressed Slabs**

408.6.1.1 A minimum area of flexural reinforcement, $A_{s,min}$ shall be provided near the tension face in the direction of the span under consideration in accordance with Table 408.6.1.1.

Table 408.6.1.1
 $A_{s,min}$ for Non-Prestressed Two-Way Slabs

Reinforcement type	f_y , MPa	$A_{s,min}$ mm ²	
Deformed bars	< 420	0.0020 A_g	
Deformed bars or welded wire reinforcement	≥ 420	Greater of:	$\frac{0.0018 \times 420}{f_y} A_g$
			0.0014 A_g

408.6.2 Minimum Flexural Reinforcement in Prestressed Slabs

408.6.2.1 For prestressed slabs, the effective prestress force $A_{ps}f_{se}$ shall provide a minimum average compressive stress of 0.9 MPa on the slab section tributary to the tendon or tendon group. For slabs with varying cross section along the slab span, either parallel or perpendicular to the tendon or tendon group, the minimum average effective prestress of 0.9 MPa is required at every cross section tributary to the tendon or tendon group along the span.

408.6.2.2 For slabs with bonded prestressed reinforcement, total quantity of A_s and A_{ps} shall be adequate to develop a factored load at least 1.2 times the cracking load calculated on the basis of f_r defined in Section 419.2.3.

408.6.2.2.1 For slabs with both flexural and shear design strength at least twice the required strength, Section 408.6.2.2 need not be satisfied.

408.6.2.3 For prestressed slabs, a minimum area of bonded deformed longitudinal reinforcement, $A_{s,min}$, shall be provided in the pre-compressed tensile zone in the direction of the span under consideration in accordance with Table 408.6.2.3.

Table 408.6.2.3
Minimum Bonded Deformed Longitudinal Reinforcement $A_{s,min}$ in Two-Way Slabs with Bonded or Unbonded Tendons

Region	Calculated f_t after all losses, MPa	$A_{s,min}$ mm ²	
Positive Moment	$f_t \leq 0.17\sqrt{f'_c}$	Not required	(a)
	$0.17\sqrt{f'_c} < f_t \leq 0.5\sqrt{f'_c}$	$\frac{N_c}{0.5f_y}$	(b) ^{[1],[2],[4]}
Negative moment at columns	$f_t \leq 0.5\sqrt{f'_c}$	0.00075 A_{cf}	(c) ^{[3],[4]}

^[1] The value of f_y shall not exceed 420 MPa.

^[2] N_c = the resultant tensile force acting on the portion of the concrete cross section that is subjected to tensile stresses due to the combined effects of service loads and effective prestress.

^[3] A_{cf} = greater gross cross-sectional area of the slab beam strips of the two orthogonal equivalent frames intersecting at a column of a two-way slab.

^[4] For slabs with bonded tendons, it shall be permitted to reduce $A_{s,min}$ by the area of the bonded prestressed reinforcement located within the area used to determine N_c for positive moment, or within the width of slab defined in Section 408.7.5.3(a) for negative moment.

408.7 Reinforcement Detailing**408.7.1 General**

408.7.1.1 Concrete cover for reinforcement shall be in accordance with Section 420.6.1.

408.7.1.2 Development lengths of deformed and prestressed reinforcement shall be in accordance with Section 425.4.

408.7.1.3 Splice lengths of deformed reinforcement shall be in accordance with Section 425.5.

408.7.1.4 Bundled bars shall be detailed in accordance with Section 425.6.

408.7.2 Flexural Reinforcement Spacing

408.7.2.1 Minimum spacing s shall be in accordance with Section 425.2.

408.7.2.2 For non-prestressed solid slabs, maximum spacing s of deformed longitudinal reinforcement shall be the lesser of **2h** and 450 mm at critical sections, and the lesser of **3h** and 450 mm at other sections.

408.7.2.3 For prestressed slabs with uniformly distributed loads, maximum spacing s of tendons or

groups of tendons in at least one direction shall be the lesser of $8h$ and 1.5 m.

408.7.2.4 Concentrated loads and openings shall be considered in determining tendon spacing.

408.7.3 Corner Restraint in Slabs

408.7.3.1 At exterior corners of slabs supported by edge walls or where one or more edge beams have a value of f greater than 1.0, reinforcement at top and bottom of slab shall be designed to resist M_u per unit width due to corner effects equal to the maximum positive M_u per unit width in the slab panel.

408.7.3.1.1 Factored moment due to corner effects, M_u , shall be assumed to be about an axis perpendicular to the diagonal from the corner in the top of the slab and about an axis parallel to the diagonal from the corner in the bottom of the slab.

408.7.3.1.2 Reinforcement shall be provided for a distance in each direction from the corner equal to 1/5 the longer span.

408.7.3.1.3 Reinforcement shall be placed parallel to the diagonal in the top of the slab and perpendicular to the diagonal in the bottom of the slab. Alternatively, reinforcement shall be placed in two layers parallel to the sides of the slab in both the top and bottom of the slab.

408.7.4 Flexural Reinforcement in Non-Prestressed Slabs

408.7.4.1 Termination of Reinforcement

408.7.4.1.1 Where a slab is supported on spandrel beams, columns, or walls, anchorage of reinforcement perpendicular to a discontinuous edge shall satisfy (a) and (b):

- a. Positive moment reinforcement shall extend to the edge of slab and have embedment, straight or hooked, at least 150 mm. into spandrel beams, columns, or walls;
- b. Negative moment reinforcement shall be bent, hooked, or otherwise anchored into spandrel beams, columns, or walls, and shall be developed at the face of support.

408.7.4.1.2 Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, anchorage of reinforcement shall be permitted within the slab.

408.7.4.1.3 For slabs without beams, reinforcement extensions shall be in accordance with (a) through (c):

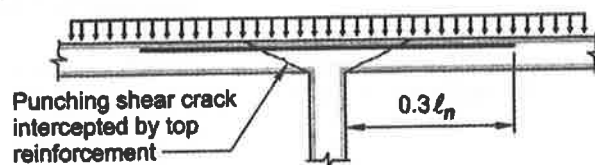
- a. Reinforcement lengths shall be at least in accordance with Figure 408.7.4.1.3(a), and if slabs act as primary members resisting lateral loads, reinforcement lengths shall be at least those required by analysis;
- b. If adjacent spans are unequal, extensions of negative moment reinforcement beyond the face of support in accordance with Figure 408.7.4.1.3(a) shall be based on the longer span;
- c. Bent bars shall be permitted only where the depth-to-span ratio permits use of bends of 45 degrees or less.

408.7.4.2 Structural Integrity

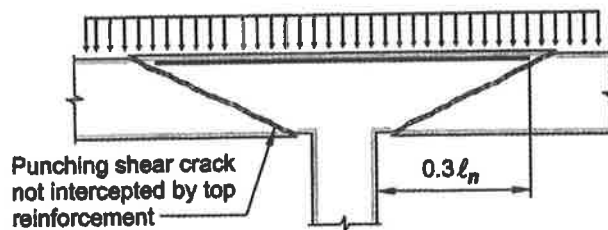
408.7.4.2.1 All bottom deformed bars or deformed wires within the column strip, in each direction, shall be continuous or spliced with full mechanical, full welded, or Class B tension splices. Splices shall be located in accordance with Figure 408.7.4.1.3(a).

STRIP	LOCATION	MINIMUM A_s AT SECTION	WITHOUT DROP PANELS	WITH DROP PANELS
COLUMN STRIP	TOP	50% REMAINDER		
	BOTTOM	100%		
MIDDLE STRIP	TOP	100%		
	BOTTOM	50% REMAINDER		
			 Exterior support (No slab continuity)	 Interior support (Continuity provided)
			 Exterior support (No slab continuity)	

Figure 408.7.4.1.3(a)
Minimum Extensions for Deformed Reinforcement in Two-Way Slabs without Beams



(a) Slab of normal proportions



(b) Thick slab

Figure 408.7.4.1.3(b)
Punching Shear Cracks in Slabs with Reinforcement Extensions Consistent with Figure 408.7.4.1.3(a)

408.7.4.2.2 At least two of the column strip bottom bars or wires in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at exterior supports.

408.7.4.2.3 In slabs with shearheads where it is not practical to pass the bottom bars through the column in accordance with Section 408.7.4.2.2, at least two bottom bars or wires in each direction shall pass through the shearhead as close to the column as practicable and be continuous or spliced with full mechanical, full welded, or Class B tension splices. At exterior columns, the bars or wires shall be anchored at the shearhead.

408.7.5 Flexural Reinforcement in Prestressed Slabs

408.7.5.1 External tendons shall be attached to the slab in a manner that maintains the specified eccentricity between the tendons and the concrete centroid through the full range of anticipated member deflections.

408.7.5.2 If bonded deformed longitudinal reinforcement is required to satisfy flexural strength or for tensile stress conditions in accordance with Eq. 408.6.2.3(b), the detailing requirements of Section 407.7.3 shall be satisfied.

408.7.5.3 Bonded longitudinal reinforcement required by Eq. 408.6.2.3(c) shall be placed in the top of the slab, and shall be in accordance with (a) through (c):

- Reinforcement shall be distributed between lines that are $1.5h$ outside opposite faces of the column support;
- At least four deformed bars, deformed wires, or bonded strands shall be provided in each direction;
- Maximum spacing s between bonded longitudinal reinforcement shall not exceed 300 mm.

408.7.5.4 Termination of Prestressed Reinforcement

408.7.5.4.1 Post-tensioned anchorage zones shall be designed and detailed in accordance with Section 425.9.

408.7.5.4.2 Post-tensioning anchorages and couplers shall be designed and detailed in accordance with Section 425.8.

408.7.5.5 Termination of Deformed Reinforcement in Slabs with Unbonded Tendons

408.7.5.5.1 Length of deformed reinforcement required by Section 408.6.2.3 shall be in accordance with (a) and (b):

- In positive moment areas, length of reinforcement shall be at least $\ell_n/3$ and be centered in those areas;
- In negative moment areas, reinforcement shall extend at least $\ell_n/6$ on each side of the face of support.

408.7.5.6 Structural Integrity

408.7.5.6.1 Except as permitted in Section 408.7.5.6.3, at least two tendons with 12 mm. diameter or larger strand shall be placed in each direction at columns in accordance with (a) or (b):

- Tendons shall pass through the region bounded by the longitudinal reinforcement of the column;
- Tendons shall be anchored within the region bounded by the longitudinal reinforcement of the column, and the anchorage shall be located beyond the column centroid and away from the anchored span.

408.7.5.6.2 Outside of the column and shear cap faces, the two structural integrity tendons required by Section 408.7.5.6.1 shall pass under any orthogonal tendons in adjacent spans.

408.7.5.6.3 Slabs with tendons not satisfying Section 408.7.5.6.1 shall be permitted if bonded bottom deformed reinforcement is provided in each direction in accordance with Sections 408.7.5.6.3.1 through 408.7.5.6.3.3.

408.7.5.6.3.1 Minimum bottom deformed reinforcement A_s in each direction shall be the greater of (a) and (b):

$$a. \quad A_s = \frac{0.37\sqrt{f'_c}b_w d}{f_y} \quad (408.7.5.6.3.1a)$$

$$b. \quad A_s = \frac{2.1b_w d}{f_y} \quad (408.7.5.6.3.1b)$$

where b_w is the width of the column face through which the reinforcement passes.

408.7.5.6.3.2 Bottom deformed reinforcement calculated in Section 408.7.5.6.3.1 shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at exterior supports.

408.7.5.6.3.3 Bottom deformed reinforcement shall be anchored to develop f_y beyond the column or shear cap face.

408.7.6 Shear Reinforcement - Stirrups

408.7.6.1 Single-leg, simple-U, multiple-U, and closed stirrups shall be permitted as shear reinforcement.

408.7.6.2 Stirrup anchorage and geometry shall be in accordance with Section 425.7.1.

408.7.6.3 If stirrups are provided, location and spacing shall be in accordance with Table 408.7.6.3.

Table 408.7.6.3
First Stirrup Location and Spacing Limits

Direction of measurement	Description of measurement	Maximum distance or spacing, mm.
Perpendicular to column face	Distance from column face to first stirrup	$d/2$
	Spacing between stirrups	$d/2$
Parallel to column face	Spacing between vertical legs of stirrups	$2d$

408.7.7 Shear Reinforcement - Headed Studs

408.7.7.1 Headed shear stud reinforcement shall be permitted if placed perpendicular to the plane of the slab.

408.7.7.1.1 The overall height of the shear stud assembly shall be at least the thickness of the slab minus the sum of (a) through (c):

- Concrete cover on the top flexural reinforcement;
- Concrete cover on the base rail;
- 1/2 the bar diameter of the flexural tension reinforcement.

408.7.7.1.2 Headed shear stud reinforcement location and spacing shall be in accordance with Table 408.7.7.1.2.

Table 408.7.7.1.2
Shear Stud Location and Spacing Limits

Direction of measurement	Description of measurement	Condition		Maximum distance or spacing, mm.
Perpendicular to column face	Distance from column face to first peripheral line of shear studs	All		$d/2$
	Constant Spacing between peripheral lines of shear studs	Non prestressed slab with	$v_u \leq \phi 0.5\sqrt{f'_c}$	$3d/4$
		Non prestressed slab with	$v_u > \phi 0.5\sqrt{f'_c}$	$d/2$
		Prestressed slabs conforming to Section 422.6.5.4		$3d/4$
Parallel to column face	Spacing between adjacent shear studs on peripheral line nearest to column face	All		$2d$

408.8 Non-Prestressed Two-Way Joist Systems

408.8.1 General

408.8.1.1 Non-prestressed two-way joist construction consists of a monolithic combination of regularly spaced ribs and a top slab designed to span in two orthogonal directions.

408.8.1.2 Width of ribs shall be at least 100 mm. at any location along the depth.

408.8.1.3 Overall depth of ribs shall not exceed 3.5 times the minimum width.

408.8.1.4 Clear spacing between ribs shall not exceed 750 mm.

408.8.1.5 V_c shall be permitted to be taken as 1.1 times the values calculated in Section 422.5.

408.8.1.6 For structural integrity, at least one bottom bar in each joist shall be continuous and shall be anchored to develop f_y at the face of supports.

408.8.1.7 Reinforcement area perpendicular to the ribs shall satisfy slab moment strength requirements, considering load concentrations, and shall be at least the shrinkage and temperature reinforcement area in accordance with Section 424.4.

408.8.1.8 Two-way joist construction not satisfying the limitations of Sections 408.8.1.1 through 408.8.1.4 shall be designed as slabs and beams.

408.8.2 Joist Systems with Structural Fillers

408.8.2.1 If permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to f'_c in the joists are used, Sections 408.8.2.1.1 and 408.8.2.1.2 shall apply.

408.8.2.1.1 Slab thickness over fillers shall be at least the greater of 1/12 the clear distance between ribs and 40 mm.

408.8.2.1.2 For calculation of shear and negative moment strength, it shall be permitted to include the vertical shells of fillers in contact with the ribs. Other portions of fillers shall not be included in strength calculations.

408.8.3 Joist Systems with Other Fillers

408.8.3.1 If fillers not complying with Section 408.8.2.1 or removable forms are used, slab thickness shall be at least the greater of 1/12 the clear distance between ribs and 50 mm.

408.9 Lift-slab Construction

408.9.1 In slabs constructed with lift-slab methods where it is impractical to pass the tendons required by Section 408.7.5.6.1 or the bottom bars required by Section 408.7.4.2 or 408.7.5.6.3 through the column, at least two post-tensioned tendons or two bonded bottom bars or wires in each direction shall pass through the lifting collar as close to the column as practicable, and be continuous or spliced with full mechanical, full welded, or Class B tension splices. At exterior columns, the reinforcement shall be anchored at the lifting collar.

408.10 Direct Design Method

408.10.1 General

408.10.1.1 Two-way slabs satisfying the limits in Section 408.10.2 shall be permitted to be designed in accordance with this section.

408.10.1.2 Variations from the limitations in Section 408.10.2 shall be permitted if demonstrated by analysis that equilibrium and geometric compatibility are satisfied, the design strength at every section is at least equal to the required strength, and serviceability conditions, including limits on deflection, are met.

408.10.1.3 Circular or regular polygon-shaped supports shall be treated as square supports with the same area.

408.10.2 Limitations for Use of Direct Design Method

408.10.2.1 There shall be at least three continuous spans in each direction.

408.10.2.2 Successive span lengths measured center-to-center of supports in each direction shall not differ by more than one-third the longer span.

408.10.2.3 Panels shall be rectangular, with the ratio of longer to shorter panel dimensions, measured center to center of supports, not to exceed 2.

408.10.2.4 Column offset shall not exceed 10 percent of the span in direction of offset from either axis between centerlines of successive columns.

408.10.2.5 All loads shall be due to gravity only and uniformly distributed over an entire panel.

408.10.2.6 Unfactored live load shall not exceed two times the unfactored dead load.

408.10.2.7 For a panel with beams between supports on all sides, Eq. 408.10.2.7a shall be satisfied for beams in the two perpendicular directions.

$$0.2 \leq \frac{\alpha_{f1} \ell_2^2}{\alpha_{f2} \ell_1^2} \leq 5.0 \quad (408.10.2.7a)$$

where α_{f1} and α_{f2} are calculated by

$$\alpha_f = \frac{E_{cb} I_b}{E_{cs} I_s} \quad (408.10.2.7b)$$

408.10.3 Total Factored Static Moment for a Span

408.10.3.1 Total factored static moment, M_o , for a span shall be calculated for a strip bounded laterally by the panel centerline on each side of the centerline of supports.

408.10.3.2 The absolute sum of positive and average negative M_u in each direction shall be at least:

$$M_o = \frac{q_u \ell_2 \ell_n^2}{8} \quad (408.10.3.2)$$

408.10.3.2.1 In Eq. 408.10.3.2, ℓ_n is the clear span length in the direction that moments are considered, shall extend from face to face of columns, capitals, brackets, or walls, and shall be at least $0.65\ell_1$.

408.10.3.2.2 In Eq. 408.10.3.2, if the transverse span of panels on either side of the centerline of supports varies, ℓ_2 shall be taken as the average of adjacent transverse spans.

408.10.3.2.3 In Eq. 408.10.3.2, if the span adjacent and parallel to a slab edge is being considered, the distance from edge to panel centerline shall be substituted for ℓ_2 .

408.10.4 Distribution of Total Factored Static Moment

408.10.4.1 In an interior span, M_o shall be distributed as follows: $0.65M_o$ to negative moment and $0.35M_o$ to positive moment.

408.10.4.2 In an end span, M_o shall be distributed in accordance with Table 408.10.4.2.

Table 408.10.4.2
Distribution Coefficients for End Spans

	Exterior edge unrestrained	Slab with beams between all supports	Slab without beams between interior supports		Exterior edge fully restrained
			Without edge beam	With edge beam	
Interior negative	0.75	0.70	0.70	0.70	0.65
Positive	0.63	0.57	0.52	0.50	0.35
Exterior negative	0	0.16	0.26	0.30	0.65

408.10.4.3 Modification of negative and positive factored moments by up to 10 percent shall be permitted if the total factored static moment for a panel, M_o , in the direction considered is at least that calculated by Eq. 408.10.3.2. Moment redistribution in accordance with Section 406.6.5 is not permitted.

408.10.4.4 Critical section for negative M_u shall be at the face of rectangular supports.

408.10.4.5 Negative M_u shall be the greater of the two interior negative M_u calculated for spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with stiffnesses of adjoining elements.

408.10.4.6 Edge beams or edges of slabs shall be designed to resist in torsion their share of exterior negative M_u .

408.10.5 Factored Moments in Column Strips

408.10.5.1 The column strip shall resist the portion of interior negative M_u in accordance with Table 408.10.5.1.

Table 408.10.5.1
Portion of Interior Negative M_u in Column Strip

$\alpha_{f1} \ell_2 / \ell_1$	ℓ_2 / ℓ_1		
	0.5	1.0	2.0
0	0.75	0.75	0.75
≥ 1.0	0.90	0.75	0.45

Note: Linear interpolations shall be made between values shown.

408.10.5.2 The column strip shall resist the portion of exterior negative M_u in accordance with Table 408.10.5.2.

Table 408.10.5.2
Portion of Exterior Negative M_u in Column Strip

$\alpha_{f1} \ell_2 / \ell_1$	β_t	ℓ_2 / ℓ_1		
		0.5	1.0	2.0
0	0	1.0	1.0	1.0
	≥ 2.5	0.75	0.75	0.75
≥ 1.0	0	1.0	1.0	1.0
	≥ 2.5	0.90	0.75	0.45

Note: Linear interpolations shall be made between values shown. β_t is calculated using Eq. 408.10.5.2a where C is calculated using Eq. 408.10.5.2b.

$$\beta_t = \frac{E_{cb} C}{2E_{cs} I_s} \quad (408.10.5.2a)$$

$$C = \Sigma \left(1 - 0.63 \frac{x}{y} \right) \frac{x^3 y}{3} \quad (408.10.5.2b)$$

408.10.5.3 For T- or L-sections, it shall be permitted to calculate the constant C in Eq. 408.10.5.2b by dividing the section, as given in Section 408.4.1.8, into separate rectangular parts and summing the values of C for each part.

408.10.5.4 If the width of the column or wall is at least $(3/4)\ell_2$, negative M_u shall be uniformly distributed across ℓ_2 .

408.10.5.5 The column strip shall resist the portion of positive M_u in accordance with Table 408.10.5.5.

Table 408.10.5.5
Portion of Positive M_u in Column Strip

$\alpha_{f1} \ell_2 / \ell_1$	ℓ_2 / ℓ_1		
	0.5	1.0	2.0
0	0.60	0.60	0.60
≥ 1.0	0.90	0.75	0.45

Note: Linear interpolations shall be made between values shown.

408.10.5.6 For slabs with beams between supports, the slab portion of column strips shall resist column strip moments not resisted by beams.

408.10.5.7 Factored Moments in Beams

408.10.5.7.1 Beams between supports shall resist the portion of column strip M_u in accordance with Table 408.10.5.7.1.

Table 408.10.5.7.1
Portion of Column Strip M_u in Beams

$\alpha_{f1} \ell_2 / \ell_1$	Distribution coefficient
0	0
≥ 1.0	0.85

Note: Linear interpolation shall be made between values shown.

408.10.5.7.2 In addition to moments calculated according to Section 408.10.5.7.1, beams shall resist moments caused by factored loads applied directly to the beams, including the weight of the beam stem above and below the slab.

408.10.6 Factored Moments in Middle Strips

408.10.6.1 That portion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to corresponding half middle strips.

408.10.6.2 Each middle strip shall resist the sum of the moments assigned to its two half middle strips.

408.10.6.3 A middle strip adjacent and parallel to a wall supported edge shall resist twice the moment assigned to the half middle strip corresponding to the first row of interior supports.

408.10.7 Factored Moments in Columns and Walls

408.10.7.1 Columns and walls built integrally with a slab system shall resist moments caused by factored loads on the slab system.

408.10.7.2 At an interior support, columns or walls above and below the slab shall resist the factored moment calculated by Eq. 408.10.7.2 in direct proportion to their stiffnesses unless a general analysis is made.

$$M_{sc} = 0.07[(q_{Du} + 0.5q_{Lu})\ell_2\ell_n^2 - q_{Du}'\ell_2'(\ell_n')^2] \quad (408.10.7.2)$$

where q_{Du}' , ℓ_2' , and ℓ_n' , refer to the shorter span.

408.10.7.3 The gravity load moment to be transferred between slab and edge column in accordance with Section 408.4.2.3 shall not be less than $0.3M_o$.

408.10.8 Factored Shear in Slab Systems with Beams

408.10.8.1 Beams between supports shall resist the portion of shear in accordance with Table 408.10.8.1 caused by factored loads on tributary areas in accordance with Figure 408.10.8.1.

Table 408.10.8.1 Portion of Shear Resisted by Beam

$(\alpha_{f1} \ell_2 / \ell_1)$	Distribution Coefficient
0	0
≥ 1.0	1.0

Note: Linear interpolation shall be made between values shown.

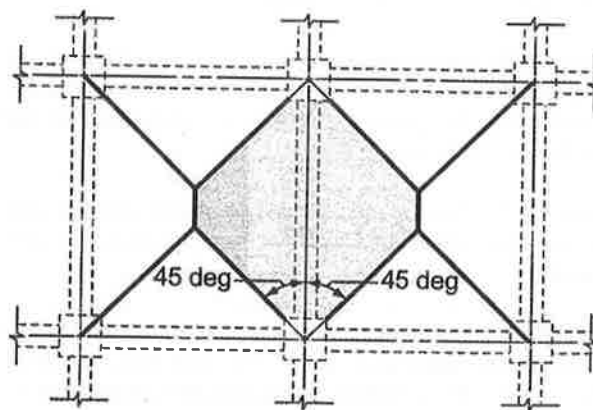


Figure 408.10.8.1
Tributary Area for Shear on an Interior Beam

408.10.8.2 In addition to shears calculated according to Section 408.10.8.1, beams shall resist shears caused by factored loads applied directly to the beams, including the weight of the beam stem above and below the slab.

408.10.8.3 Calculation of required slab shear strength based on the assumption that loads is distributed to supporting beams in accordance with Section 408.10.8.1 shall be permitted. Shear resistance to total V_u occurring on a panel shall be provided.

408.11 Equivalent Frame Method

408.11.1 General

408.11.1.1 All sections of slabs and supporting members in two-way slab systems designed by the equivalent frame method shall resist moments and shears obtained from an analysis in accordance with Sections 408.11.2 through 408.11.6.

408.11.1.2 Live load shall be arranged in accordance with Section 406.4.3.

408.11.1.3 It shall be permitted to account for the contribution of metal column capitals to stiffness, resistance to moment, and resistance to shear.

408.11.1.4 It shall be permitted to neglect the change in length of columns and slabs due to direct stress, and deflections due to shear.

408.11.2 Equivalent Frames

408.11.2.1 The structure shall be modeled by equivalent frames on column lines taken longitudinally and transversely through the building.

408.11.2.2 Each equivalent frame shall consist of a row of columns or supports and slab-beam strips bounded laterally by the panel centerline on each side of the centerline of columns or supports.

408.11.2.3 Frames adjacent and parallel to an edge shall be bounded by that edge and the centerline of the adjacent panel.

408.11.2.4 Columns or supports shall be assumed to be attached to slab-beam strips by torsional members transverse to the direction of the span for which moments are being calculated and extending to the panel centerlines on each side of a column.

408.11.2.5 Analysis of each equivalent frame in its entirety shall be permitted. Alternatively, for gravity

loading, a separate analysis of each floor or roof with the far ends of columns considered fixed is permitted.

408.11.2.6 If slab-beams are analyzed separately, it shall be permitted to calculate the moment at a given support by assuming that the slab-beam is fixed at supports two or more panels away, provided the slab continues beyond the assumed fixed supports.

408.11.3 Slab-Beams

408.11.3.1 The moment of inertia of slab-beams from the center of the column to the face of the column, bracket, or capital shall be assumed equal to the moment of inertia of the slab-beam at the face of the column, bracket, or capital divided by the quantity $(1 - c_2/\ell_2)^2$, where c_2 and ℓ_2 are measured transverse to the direction of the span for which moments are being determined.

408.11.3.2 Variation in moment of inertia along the axis of slab-beams shall be taken into account.

408.11.3.3 It shall be permitted to use the gross cross-sectional area of concrete to determine the moment of inertia of slab-beams at any cross section outside of joints or column capitals.

408.11.4 Columns

408.11.4.1 The moment of inertia of columns from top to bottom of the slab-beam at a joint shall be assumed to be infinite.

408.11.4.2 Variation in moment of inertia along the axis of columns shall be taken into account.

408.11.4.3 It shall be permitted to use the gross cross-sectional area of concrete to determine the moment of inertia of columns at any cross section outside of joints or column capitals.

408.11.5 Torsional Members

408.11.5.1 Torsional members shall be assumed to have a constant cross section throughout their length consisting of the greatest of (a) through (c):

- a. A portion of slab having a width equal to that of the column, bracket, or capital in the direction of the span for which moments are being determined;
- b. For monolithic or fully composite construction, the portion of slab specified in (a) plus that part of the transverse beam above and below the slab;

- c. The transverse beam in accordance with Section 408.4.1.8.

408.11.5.2 Where beams frame into columns in the direction of the span for which moments are being calculated, the torsional stiffness shall be multiplied by the ratio of the moment of inertia of the slab with such a beam to the moment of inertia of the slab without such a beam.

408.11.6 Factored Moments

408.11.6.1 At interior supports, the critical section for negative M_u in both column and middle strips shall be taken at the face of rectilinear supports, but not farther away than $0.175\ell_1$ from the center of a column.

408.11.6.2 At exterior supports without brackets or capitals, the critical section for negative M_u in the span perpendicular to an edge shall be taken at the face of the supporting element.

408.11.6.3 At exterior supports with brackets or capitals, the critical section for negative M_u in the span perpendicular to an edge shall be taken at a distance from the face of the supporting element not exceeding one-half the projection of the bracket or capital beyond the face of the supporting element.

408.11.6.4 Circular or regular polygon-shaped supports shall be assumed to be square supports with the same area for location of critical section for negative design moment.

408.11.6.5 Where slab systems within limitations of Section 408.10.2 are analyzed by the equivalent frame method, it shall be permitted to reduce the calculated moments in such proportion that the absolute sum of the positive and average negative design moments need not exceed the value obtained from Eq. 408.10.3.2.

408.11.6.6 It shall be permitted to distribute moments at critical sections to column strips, beams, and middle strips in accordance with the direct design method in Section 408.10 provided that Eq. 408.10.2.7a is satisfied.

SECTION 409 BEAMS

409.1 Scope

409.1.1 This section shall apply to the design of non-prestressed and prestressed beams, including:

- a. Composite beams of concrete elements constructed in separate placements but connected so that all elements resist loads as a unit;
- b. One-way joist systems in accordance with Section 409.8;
- c. Deep beams in accordance with Section 409.9.

409.2 General

409.2.1 Materials

409.2.1.1 Design properties for concrete shall be selected to be in accordance with Section 419.

409.2.1.2 Design properties for steel reinforcement shall be selected to be in accordance with Section 420.

409.2.1.3 Materials, design, and detailing requirements for embedments in concrete shall be in accordance with Section 420.7.

409.2.2 Connection to Other Members

409.2.2.1 For cast-in-place construction, beam-column joints shall satisfy Section 415.

409.2.2.2 For precast construction, connections shall satisfy the force transfer requirements of Section 416.2.

409.2.3 Stability

409.2.3.1 If a beam is not continuously laterally braced, (a) and (b) shall be satisfied:

- a. Spacing of lateral bracing shall not exceed 50 times the least width of compression flange or face;
- b. Spacing of lateral bracing shall take into account effects of eccentric loads.

409.2.3.2 In prestressed beams, buckling of thin webs and flanges shall be considered. If there is intermittent contact between prestressed reinforcement and an oversize duct, member buckling between contact points shall be considered.

409.2.4 T-Beam Construction

409.2.4.1 In T-beam construction, flange and web concrete shall be placed monolithically or made composite in accordance with Section 416.4.

409.2.4.2 Effective flange width shall be in accordance with Section 406.3.2.

409.2.4.3 For T-beam flanges where the primary flexural slab reinforcement is parallel to the longitudinal axis of the beam, reinforcement in the flange perpendicular to the longitudinal axis of the beam shall be in accordance with Section 407.5.2.3.

409.2.4.4 For torsional design according to Section 422.7, the overhanging flange width used to calculate A_{cp} , A_g , and p_{cp} shall be in accordance with (a) and (b):

- The overhanging flange width shall include that portion of slab on each side of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness;
- The overhanging flanges shall be neglected in cases where the parameter A_p^2/p_{cp} for solid sections or A_p^2/p_{cp} for hollow sections calculated for a beam with flanges is less than that calculated for the same beam ignoring the flanges.

409.3 Design Limits**409.3.1 Minimum Beam Depth**

409.3.1.1 For non-prestressed beams not supporting or attached to partitions or other construction likely to be damaged by large deflections, overall beam depth h shall satisfy the limits in Table 409.3.1.1, unless the calculated deflection limits of Section 409.3.2 are satisfied.

Table 409.3.1.1
Minimum Depth of Non-Prestressed Beams

Support Condition	Minimum $h^{[1]}$
Simply supported	$\ell/16$
One end continuous	$\ell/18.5$
Both ends continuous	$\ell/21$
Cantilever	$\ell/8$

^[1] Expressions applicable for normal weight concrete and $f_y = 420$ MPa. For other cases, minimum h shall be modified in accordance with Sections 409.3.1.1.1 through 409.3.1.1.3, as appropriate.

409.3.1.1.1 For f_y other than 420 MPa, the expressions in Table 409.3.1.1 shall be multiplied by $(0.4 + f_y/700)$.

409.3.1.1.2 For non-prestressed beams made of lightweight concrete having w_c in the range of 1440 to 1840 kg/m³, the expressions in Table 409.3.1.1 shall be multiplied by the greater of (a) and (b):

- $1.65 - 0.0003w_c$;
- 1.09.

409.3.1.1.3 For non-prestressed composite beams made of a combination of lightweight and normal-weight concrete shored during construction, and where the lightweight concrete is in compression, the modifier of Section 409.3.1.1.2 shall apply.

409.3.1.2 The thickness of a concrete floor finish shall be permitted to be included in h if it is placed monolithically with the beam, or if the floor finish is designed to be composite with the beam in accordance with Section 416.4.

409.3.2 Calculated Deflection Limits

409.3.2.1 For non-prestressed beams not satisfying Section 409.3.1 and for prestressed beams, immediate and time-dependent deflections shall be calculated in accordance with Section 424.2 and shall not exceed the limits in Section 424.2.2.

409.3.2.2 For non-prestressed composite concrete beams satisfying Section 409.3.1, deflections occurring after the member becomes composite need not be calculated. Deflections occurring before the member becomes composite shall be investigated unless the pre-composite depth also satisfies Section 409.3.1.

409.3.3 Reinforcement Strain Limit in Non-Prestressed Beams

409.3.3.1 For non-prestressed beams with $P_u < 0.10f'_cA_g$, ϵ_t shall be at least 0.004.

409.3.4 Stress Limits in Prestressed Beams

409.3.4.1 Prestressed beams shall be classified as Class U, T, or C in accordance with Section 424.5.2.

409.3.4.2 Stresses in prestressed beams immediately after transfer and at service loads shall not exceed permissible stresses in Sections 424.5.3 and 424.5.4.

409.4 Required Strength

409.4.1 General

409.4.1.1 Required strength shall be calculated in accordance with the factored load combinations in Section 405.

409.4.1.2 Required strength shall be calculated in accordance with the analysis procedures in Section 406.

409.4.1.3 For prestressed beams, effects of reactions induced by prestressing shall be considered in accordance with Section 405.3.11.

409.4.2 Factored Moment

409.4.2.1 For beams built integrally with supports, M_u at the support shall be permitted to be calculated at the face of support.

409.4.3 Factored Shear

409.4.3.1 For beams built integrally with supports, V_u at the support shall be permitted to be calculated at the face of support.

409.4.3.2 Sections between the face of support and a critical section located d from the face of support for non-prestressed beams and $h/2$ from the face of support for prestressed beams shall be permitted to be designed for V_u at that critical section if (a) through (c) are satisfied:

- Support reaction, in direction of applied shear, introduces compression into the end region of the beam;
- Loads are applied at or near the top surface of the beam;
- No concentrated load occurs between the face of support and critical section.

409.4.4 Factored Torsion

409.4.4.1 Unless determined by a more detailed analysis, it shall be permitted to take the torsional loading from a slab as uniformly distributed along the beam.

409.4.4.2 For beams built integrally with supports, T_u at the support shall be permitted to be calculated at the face of support.

409.4.4.3 Sections between the face of support and a critical section located d from the face of support for non-prestressed beams or $h/2$ from the face of support for

prestressed beams shall be permitted to be designed for T_u at that critical section unless a concentrated torsional moment occurs within this distance. In that case, the critical section shall be taken at the face of the support.

409.4.4.4 It shall be permitted to reduce T_u in accordance with Section 422.7.3.

409.5 Design Strength

409.5.1 General

409.5.1.1 For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$ including (a) through (d). Interaction between load effects shall be considered.

a. $\phi M_n \geq M_u$

b. $\phi V_n \geq V_u$

c. $\phi T_n \geq T_u$

d. $\phi P_n \geq P_u$

409.5.1.2 ϕ shall be determined in accordance with Section 421.2.

409.5.2 Moment

409.5.2.1 If $P_u < 0.10f'_cA_g$, M_n shall be calculated in accordance with Section 422.3.

409.5.2.2 If $P_u \geq 0.10f'_cA_g$, M_n shall be calculated in accordance with Section 422.4.

409.5.2.3 For prestressed beams, external tendons shall be considered as unbonded tendons in calculating flexural strength, unless the external tendons are effectively bonded to the concrete along the entire length.

409.5.3 Shear

409.5.3.1 V_n shall be calculated in accordance with Section 422.5.

409.5.3.2 For composite concrete beams, horizontal shear strength V_{nh} shall be calculated in accordance with Section 416.4.

409.5.4 Torsion

409.5.4.1 If $T_u < \phi T_{th}$ where T_{th} is given in Section 422.7, it shall be permitted to neglect torsional effects. The minimum reinforcement requirements of Section

409.6.4 and the detailing requirements of Sections 409.7.5 and 409.7.6.3 need not be satisfied.

409.5.4.2 T_n shall be calculated in accordance with Section 422.7.

409.5.4.3 Longitudinal and transverse reinforcement required for torsion shall be added to that required for the V_u , M_u , and P_u that act in combination with the torsion.

409.5.4.4 For prestressed beams, the total area of longitudinal reinforcement, A_s and A_{ps} , at each section shall be designed to resist M_u at that section, plus an additional concentric longitudinal tensile force equal to $A_t f_y$, based on T_u at that section.

409.5.4.5 It shall be permitted to reduce the area of longitudinal torsional reinforcement in the flexural compression zone by an amount equal to $M_u / (0.9 d f_y)$, where M_u occurs simultaneously with T_u at that section, except that the longitudinal reinforcement area shall not be less than the minimum required in Section 409.6.4.

409.5.4.6 For solid sections with an aspect ratio, $h/b_t \geq 4.5$, it shall be permitted to use an alternative design procedure, provided the adequacy of the procedure has been shown by analysis and substantial agreement with results of comprehensive tests. The minimum reinforcement requirements of Section 409.6.4 need not be satisfied, but the detailing requirements of Sections 409.7.5 and 409.7.6.3 apply.

409.5.4.7 For solid precast sections with an aspect ratio $h/b_t \geq 4.5$, it shall be permitted to use an alternative design procedure and open web reinforcement, provided the adequacy of the procedure and reinforcement have been shown by analysis and substantial agreement with results of comprehensive tests. The minimum reinforcement requirements of Section 409.6.4 and detailing requirements of Sections 409.7.5 and 409.7.6.3 need not be satisfied.

409.6 Reinforcement Limits

409.6.1 Minimum Flexural Reinforcement in Non-Prestressed Beams

409.6.1.1 A minimum area of flexural reinforcement $A_{s,min}$ shall be provided at every section where tension reinforcement is required by analysis.

409.6.1.2 $A_{s,min}$ shall be the greater of (a) and (b), except as provided in Section 409.6.1.3. For a statically determinate beam with a flange in tension, the value of b_w shall be the lesser of b_f and $2b_w$.

$$\begin{aligned} \text{a. } & \frac{0.25 \sqrt{f'_c}}{f_y} b_w d \\ \text{b. } & \frac{1.4}{f_y} b_w d \end{aligned}$$

409.6.1.3 If A_s provided at every section is at least one-third greater than $A_{s,min}$ required by analysis, Sections 409.6.1.1 and 409.6.1.2 need not be satisfied.

409.6.2 Minimum Flexural Reinforcement in Prestressed Beams

409.6.2.1 For beams with bonded prestressed reinforcement, total quantity of A_s and A_{ps} shall be adequate to develop a factored load at least 1.2 times the cracking load calculated on the basis of f_r defined in Section 419.2.3.

409.6.2.2 For beams with both flexural and shear design strength at least twice the required strength, Section 409.6.2.1 need not be satisfied.

409.6.2.3 For beams with unbonded tendons, the minimum area of bonded deformed longitudinal reinforcement $A_{s,min}$ shall be:

$$A_{s,min} = 0.004 A_{ct} \quad (409.6.2.3)$$

where A_{ct} is the area of that part of the cross section between the flexural tension face and the centroid of the gross section.

409.6.3 Minimum Shear Reinforcement

409.6.3.1 A minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all regions where $V_u > 0.5 \phi V_c$ except for the cases in Table 409.6.3.1, where at least $A_{v,min}$ shall be provided where $V_u > \phi V_c$.

409.6.3.2 If shown by testing that the required M_n and V_n can be developed, Section 409.6.3.1 need not be satisfied. Such tests shall simulate effects of differential settlement, creep, shrinkage, and temperature change, based on a realistic assessment of these effects occurring in service.

409.6.3.3 If shear reinforcement is required and torsional effects can be neglected according to Section 409.5.4.1, $A_{v,min}$ shall be in accordance with Table 409.6.3.3

Table 409.6.3.1
Cases Where $A_{v,min}$ is not Required
if $0.5\phi V_c < V_u \leq \phi V_c$

Beam Type	Conditions
Shallow Depth	$h \leq 250$ mm
Integral with Slab	$h \leq \text{greater of } 2.5t_f \text{ or } 0.5b_w$ and $h \leq 600$ mm
Constructed with steel fibered reinforced normal-weight concrete conforming to Sections 426.4.1.5.1(a), 426.4.2.2(d), and 426.12.5.1(a) and with $f'_c \leq 40$ MPa	$h \leq 600$ mm and $V_u \leq 0.17\phi\sqrt{f'_c b_w d}$
One-way joist system	in accordance with Section 409.8

Table 409.6.3.3 Required $A_{v,min}$

Beam Type	$A_{v,min}/S$		
Non-prestressed and prestressed with $A_{ps}f_{se} < 0.40(A_{ps} + f_{pu} + A_s + f_y)$	Greater of:	$0.062\sqrt{f'_c} \frac{b_w}{f_{yt}}$	a
		$0.35 \frac{b_w}{f_{yt}}$	b
Prestressed with $A_{ps}f_{se} \geq 0.40(A_{ps} + f_{pu} + A_s + f_y)$	Lesser of:	Greater of: $0.062\sqrt{f'_c} \frac{b_w}{f_{yt}}$	c
		$0.35 \frac{b_w}{f_{yt}}$	d
		$\frac{A_{ps}f_{pu}}{80f_{yt}d} \sqrt{\frac{d}{b_w}}$	e

409.6.4 Minimum Torsional Reinforcement

409.6.4.1 A minimum area of torsional reinforcement shall be provided in all regions where $T_u \geq \phi T_{th}$ accordance with Section 422.7.

409.6.4.2 If torsional reinforcement is required, minimum transverse reinforcement $(A_v + 2A_t)_{min}/s$ shall be the greater of (a) and (b):

a. $0.062\sqrt{f'_c} \frac{b_w}{f_{yt}}$

b. $0.35 \frac{b_w}{f_{yt}}$

409.6.4.3 If torsional reinforcement is required, minimum area of longitudinal reinforcement $A_{e,min}$ shall be the lesser of (a) and (b):

a. $\frac{0.42\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{A_t}{s}\right)p_h \frac{f_{yt}}{f_y}$

b. $\frac{0.42\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{0.175b_w}{f_{yt}}\right)p_h \frac{f_{yt}}{f_y}$

409.7 Reinforcement Detailing

409.7.1 General

409.7.1.1 Concrete cover for reinforcement shall be in accordance with Section 420.6.1.

409.7.1.2 Development lengths of deformed and prestressed reinforcement shall be in accordance with Section 425.4.

409.7.1.3 Splices of deformed reinforcement shall be in accordance with Section 425.5.

409.7.1.4 Bundled bars shall be in accordance with Section 425.6.

409.7.2 Reinforcement Spacing

409.7.2.1 Minimum spacing s shall be in accordance with Section 425.2.

409.7.2.2 For non-prestressed and Class C prestressed beams, spacing of bonded longitudinal reinforcement closest to the tension face shall not exceed s given in Section 424.3.

409.7.2.3 For non-prestressed and Class C prestressed beams with h exceeding 900 mm., longitudinal skin reinforcement shall be uniformly distributed on both side faces of the beam for a distance $h/2$ from the tension face. Spacing of skin reinforcement shall not exceed s given in Section 424.3.2, where c_c is the clear cover from the skin reinforcement to the side face. It shall be permitted to include skin reinforcement in strength calculations if a strain compatibility analysis is made.

409.7.3 Flexural Reinforcement in Non-Prestressed Beams

409.7.3.1 Calculated tensile or compressive force in reinforcement at each section of the beam shall be developed on each side of that section.

409.7.3.2 Critical locations for development of reinforcement are points of maximum stress and points along the span where bent or terminated tension reinforcement is no longer required to resist flexure.

409.7.3.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the greater of d and $12d_b$, except at supports of simply-supported spans and at free ends of cantilevers.

409.7.3.4 Continuing flexural tension reinforcement shall have an embedment length at least ℓ_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

409.7.3.5 Flexural tension reinforcement shall not be terminated in a tension zone unless (a), (b), or (c) is satisfied:

- a. $V_u \leq (2/3)\phi V_n$ at the cutoff point;
- b. For 36 mm ϕ bars and smaller, continuing reinforcement provides double the area required for flexure at the cutoff point and $V_u \leq (3/4)\phi V_n$.
- c. Stirrup or hoop area in excess of that required for shear and torsion is provided along each terminated bar or wire over a distance $3/4d$ from the termination point. Excess stirrup or hoop area shall be at least $60b_ws/f_{yt}$. Spacing s shall not exceed $d/8\beta_b$.

409.7.3.6 Adequate anchorage shall be provided for tension reinforcement where reinforcement stress is not directly proportional to moment, such as in sloped, stepped, or tapered beams; or where tension reinforcement is not parallel to the compression face.

409.7.3.7 Development of tension reinforcement by bending across the web to be anchored or made continuous with reinforcement on the opposite face of beam shall be permitted.

409.7.3.8 Termination of Reinforcement

409.7.3.8.1 At simple supports, at least one-third the maximum positive moment reinforcement shall extend along the beam bottom into the support at least 150 mm., except for precast beams where such reinforcement shall extend at least to the center of the bearing length.

409.7.3.8.2 At other supports, at least one-fourth the maximum positive moment reinforcement shall extend along the beam bottom into the support at least 150 mm.

409.7.3.8.3 At simple supports and points of inflection, d_b for positive moment tension reinforcement shall be limited such that ℓ for that reinforcement satisfies (a) or (b); If reinforcement terminates beyond the centerline of supports by a standard hook or a mechanical anchorage at least equivalent to a standard hook, (a) or (b) need not be satisfied.

- a. $\ell_d \leq (1.3M_n/V_u + \ell_a)$ if end of reinforcement is confined by a compressive reaction;
- b. $\ell_d \leq (M_n/V_u + \ell_a)$ if end of reinforcement is not confined by a compressive reaction.

Where: M_n is calculated assuming all reinforcement at the section is stressed to f_y and V_u is calculated at the section. At a support, ℓ_a is the embedment length beyond the center of the support. At a point of inflection, ℓ_a is the embedment length beyond the point of inflection limited to the greater of d and $12d_b$.

409.7.3.8.4 At least one-third of the negative moment reinforcement at a support shall have an embedment length beyond the point of inflection at least the greatest of d , $12d_b$, and $\ell_n/16$.

409.7.4 Flexural Reinforcement in Prestressed Beams

409.7.4.1 External tendons shall be attached to the member in a manner that maintains the specified eccentricity between the tendons and the concrete centroid through the full range of anticipated member deflections.

409.7.4.2 If non-prestressed reinforcement is required to satisfy flexural strength, the detailing requirements of Section 409.7.3 shall be satisfied.

409.7.4.3 Termination of Prestressed Reinforcement

409.7.4.3.1 Post-tensioned anchorage zones shall be designed and detailed in accordance with Section 425.9.

409.7.4.3.2 Post-tensioning anchorages and couplers shall be designed and detailed in accordance with Section 425.8.

409.7.4.4 Termination of deformed reinforcement in beams with unbonded tendons

409.7.4.4.1 Length of deformed reinforcement required by Section 409.6.2.3 shall be in accordance with (a) and (b):

- a. At least $\ell_n/3$ in positive moment areas and be centered in those areas;
- b. At least $\ell_n/6$ on each side of the face of support in negative moment areas.

409.7.5 Longitudinal Torsional Reinforcement

409.7.5.1 If torsional reinforcement is required, longitudinal torsional reinforcement shall be distributed around the perimeter of closed stirrups that satisfy Section 425.7.1.6, or hoops with spacing not greater than 300 mm. The longitudinal reinforcement shall be inside the stirrup or hoop and at least one longitudinal bar or tendon shall be placed in each corner.

409.7.5.2 Longitudinal torsional reinforcement shall have a diameter at least **0.042** times the transverse reinforcement spacing, but not less than 10 mm.

409.7.5.3 Longitudinal torsional reinforcement shall extend for a distance of at least $(b_t + d)$ beyond the point required by analysis.

409.7.5.4 Longitudinal torsional reinforcement shall be developed at the face of the support at both ends of the beam.

409.7.6 Transverse Reinforcement

409.7.6.1 General

409.7.6.1.1 Transverse reinforcement shall be in accordance with this section. The most restrictive requirements shall apply.

409.7.6.1.2 Details of transverse reinforcement shall be in accordance with Section 425.7.

409.7.6.2 Shear

409.7.6.2.1 If required, shear reinforcement shall be provided using stirrups, hoops, or longitudinal bent bars.

409.7.6.2.2 Maximum spacing of shear reinforcement shall be in accordance with Table 409.7.6.2.2.

409.7.6.2.3 Inclined stirrups and longitudinal bars bent to act as shear reinforcement shall be spaced so that every 45-degree line, extending $d/2$ toward the reaction from mid-depth of member to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

409.7.6.2.4 Longitudinal bars bent to act as shear reinforcement, if extended into a region of tension, shall

be continuous with longitudinal reinforcement and, if extended into a region of compression, shall be anchored $d/2$ beyond mid-depth of member.

Table 409.7.6.2.2 Maximum Spacing of Shear Reinforcement

V_s		Maximum s , mm	
		Non-prestressed beam	Prestressed beam
$\leq 0.33\sqrt{f'_c}b_wd$	Lesser of:	$\frac{d}{2}$	$\frac{3h}{4}$
		600	
$> 0.33\sqrt{f'_c}b_wd$	Lesser of:	$\frac{d}{4}$	$\frac{3h}{8}$
		300	

409.7.6.3 Torsion

409.7.6.3.1 If required, transverse torsional reinforcement shall be closed stirrups satisfying Section 425.7.1.6 or hoops.

409.7.6.3.2 Transverse torsional reinforcement shall extend a distance of at least $(b_t + d)$ beyond the point required by analysis.

409.7.6.3.3 Spacing of transverse torsional reinforcement shall not exceed the lesser of $p_h/8$ and 300 mm.

409.7.6.3.4 For hollow sections, the distance from the centerline of the transverse torsional reinforcement to the inside face of the wall of the hollow section shall be at least $0.5A_{oh}/p_h$.

409.7.6.4 Lateral Support of Compression Reinforcement

409.7.6.4.1 Transverse reinforcement shall be provided throughout the distance where longitudinal compression reinforcement is required. Lateral support of longitudinal compression reinforcement shall be provided by closed stirrups or hoops in accordance with Sections 409.7.6.4.2 through 409.7.6.4.4.

409.7.6.4.2 Size of transverse reinforcement shall be at least (a) or (b). Deformed wire or welded wire reinforcement of equivalent area shall be permitted.

- a. 10 mm ϕ for longitudinal bars 32 mm ϕ and smaller;

- b. 12 mm ϕ for longitudinal bars 36 mm ϕ and larger and for longitudinal bundled bars.

409.7.6.4.3 Spacing of transverse reinforcement shall not exceed the least of (a) through (c):

- a. $16d_b$ of longitudinal reinforcement;
- b. $48d_b$ of transverse reinforcement;
- c. Least dimension of beam.

409.7.6.4.4 Longitudinal compression reinforcement shall be arranged such that every corner and alternate compression bar shall be enclosed by the corner of the transverse reinforcement with an included angle of not more than 135 degrees, and no bar shall be farther than 150 mm. clear on each side along the transverse reinforcement from such an enclosed bar.

409.7.7 Structural Integrity Reinforcement in Cast-in-Place Beams

409.7.7.1 For beams along the perimeter of the structure, structural integrity reinforcement shall be in accordance with (a) through (c):

- a. At least one-quarter the maximum positive moment reinforcement, but not less than two bars or strands, shall be continuous;
- b. At least one-sixth the negative moment reinforcement at the support, but not less than two bars or strands, shall be continuous;
- c. Longitudinal structural integrity reinforcement shall be enclosed by closed stirrups in accordance with Section 425.7.1.6 or hoops along the clear span of the beam.

409.7.7.2 For other than perimeter beams, structural integrity reinforcement shall be in accordance with (a) or (b):

- a. At least one-quarter the maximum positive moment reinforcement, but not less than two bars or strands,, shall be continuous;
- b. Longitudinal reinforcement shall be enclosed by closed stirrups in accordance with Section 425.7.1.6 or hoops along the clear span of the beam.

409.7.7.3 Longitudinal structural integrity reinforcement shall pass through the region bounded by the longitudinal reinforcement of the column.

409.7.7.4 Longitudinal structural integrity reinforcement at non continuous supports shall be anchored to develop f_y at the face of the support.

409.7.7.5 If splices are necessary in continuous structural integrity reinforcement, the reinforcement shall be spliced in accordance with (a) and (b):

- a. Positive moment reinforcement shall be spliced at or near the support;
- b. Negative moment reinforcement shall be spliced at or near mid-span.

409.7.7.6 Splices shall be full mechanical, full welded, or Class B tension lap splices.

409.8 Non-Prestressed One-way Joist Systems

409.8.1 General

409.8.1.1 Non-prestressed one-way joist construction consists of a monolithic combination of regularly spaced ribs and a top slab designed to span in one direction.

409.8.1.2 Width of ribs shall be at least 100 mm at any location along the depth.

409.8.1.3 Overall depth of ribs shall not exceed 3.5 times the minimum width.

409.8.1.4 Clear spacing between ribs shall not exceed 750 mm.

409.8.1.5 V_c shall be permitted to be taken as 1.1 times the value calculated in Section 422.5.

409.8.1.6 For structural integrity, at least one bottom bar in each joist shall be continuous and shall be anchored to develop f_y at the face of supports.

409.8.1.7 Reinforcement perpendicular to the ribs shall be provided in the slab as required for flexure, considering load concentrations, and shall be at least that required for shrinkage and temperature in accordance with Section 424.4.

409.8.1.8 One-way joist construction not satisfying the limitations of Sections 409.8.1.1 through 409.8.1.4 shall be designed as slabs and beams.

409.8.2 Joist Systems with Structural Fillers

409.8.2.1 If permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to f'_c in the joists are used, Sections 409.8.2.1.1 and 409.8.2.1.2 shall apply.

409.8.2.1.1 Slab thickness over fillers shall be at least the greater of 1/12 the clear distance between ribs and 40 mm.

409.8.2.1.2 For calculation of shear and negative moment strength, it shall be permitted to include the vertical shells of fillers in contact with the ribs. Other portions of fillers shall not be included in strength calculations.

409.8.3 Joist Systems with Other Fillers

409.8.3.1 If fillers not complying with Section 409.8.2.1 or removable forms are used, slab thickness shall be at least the greater of 1/12 the clear distance between ribs and 50 mm.

409.9 Deep Beams

409.9.1 General

409.9.1.1 Deep beams are members that are loaded on one face and supported on the opposite face such that strut-like compression elements can develop between the loads and supports and that satisfy (a) or (b):

- Clear span does not exceed four times the overall member depth, h ;
- Concentrated loads exist within a distance $2h$ from the face of the support.

409.9.1.2 Deep beams shall be designed taking into account nonlinear distribution of longitudinal strain over the depth of the beam.

409.9.1.3 Strut-and-tie models in accordance with Section 423 are deemed to satisfy Section 409.9.1.2.

409.9.2 Dimensional Limits

409.9.2.1 Deep beam dimensions shall be selected such that:

$$V_u \leq 0.83\phi\sqrt{f'_c}b_wd \quad (409.9.2.1)$$

409.9.3 Reinforcement Limits

409.9.3.1 Distributed reinforcement along the side faces of deep beams shall be at least that required in (a) and (b):

- The area of distributed reinforcement perpendicular to the longitudinal axis of the beam, A_v , shall be at least $0.0025b_ws$, where s is the spacing of the distributed transverse reinforcement;
- The area of distributed reinforcement parallel to the longitudinal axis of the beam, A_{vh} , shall be at least $0.0025b_ws_2$, where s_2 is the spacing of the distributed longitudinal reinforcement.

409.9.3.2 The minimum area of flexural tension reinforcement, $A_{s,min}$, shall be determined in accordance with Section 409.6.1.

409.9.4 Reinforcement Detailing

409.9.4.1 Concrete cover shall be in accordance with Section 420.6.1.

409.9.4.2 Minimum spacing for longitudinal reinforcement shall be in accordance with Section 425.2.

409.9.4.3 Spacing of distributed reinforcement required in Section 409.9.3.1 shall not exceed the lesser of $d/5$ and 300 mm.

409.9.4.4 Development of tension reinforcement shall account for distribution of stress in reinforcement that is not directly proportional to the bending moment.

409.9.4.5 At simple supports, positive moment tension reinforcement shall be anchored to develop f_y at the face of the support. If a deep beam is designed using Section 423, the positive moment tension reinforcement shall be anchored in accordance with Sections 423.8.2 and 423.8.3.

409.9.4.6 At interior supports, (a) and (b) shall be satisfied:

- Negative moment tension reinforcement shall be continuous with that of the adjacent spans;
- Positive moment tension reinforcement shall be continuous or spliced with that of the adjacent spans.

SECTION 410 COLUMNS

410.1 Scope

410.1.1 This section shall apply to the design of non-prestressed, prestressed, and composite columns, including reinforced concrete pedestals.

410.1.2 Design of plain concrete pedestals shall be in accordance with Section 414.

410.2 General

410.2.1 Materials

410.2.1.1 Design properties for concrete shall be selected to be in accordance with Section 419.

410.2.1.2 Design properties for steel reinforcement and structural steel used in composite columns shall be selected to be in accordance with Section 420.

410.2.1.3 Materials, design, and detailing requirements for embedments in concrete shall be in accordance with Section 420.7.

410.2.2 Composite Columns

410.2.2.1 If a structural steel shape, pipe, or tubing is used as longitudinal reinforcement, the column shall be designed as a composite column.

410.2.3 Connection to Other Members

410.2.3.1 For cast-in-place construction, beam-column and slab-column joints shall satisfy Section 415.

410.2.3.2 For precast construction, connections shall satisfy the force transfer requirements of Section 416.2.

410.2.3.3 Connections of columns to foundations shall satisfy Section 416.3.

410.3 Design Limits

410.3.1 Dimensional Limits

410.3.1.1 For columns with a square, octagonal, or other shaped cross section, it shall be permitted to base gross area considered, required reinforcement, and design strength on a circular section with a diameter equal to the least lateral dimension of the actual shape.

410.3.1.2 For columns with cross sections larger than required by considerations of loading, it shall be permitted to base gross area considered, required reinforcement, and design strength on a reduced effective area, not less than one-half the total area. This provision shall not apply to columns in special moment frames designed in accordance with Section 418.

410.3.1.3 For columns built monolithically with a concrete wall, the outer limits of the effective cross section of the column shall not be taken greater than 40mm. outside the transverse reinforcement.

410.3.1.4 For columns with two or more interlocking spirals, outer limits of the effective cross section shall be taken at a distance outside the spirals equal to the minimum required concrete cover.

410.3.1.5 If a reduced effective area is considered according to Sections 410.3.1.1 through 410.3.1.4, structural analysis and design of other parts of the structure that interact with the column shall be based on the actual cross section.

410.3.1.6 For composite columns with a concrete core encased by structural steel, the thickness of the steel encasement shall be at least (a) or (b):

a. $b \sqrt{\frac{f_y}{3E_s}}$ for each face of width b ;

b. $h \sqrt{\frac{f_y}{8E_s}}$ for circular sections of diameter h .

410.4 Required Strength

410.4.1 General

410.4.1.1 Required strength shall be calculated in accordance with the factored load combinations in Section 405.

410.4.1.2 Required strength shall be calculated in accordance with the analysis procedures in Section 406.

410.4.2 Factored Axial Force and Moment

410.4.2.1 P_u and M_u occurring simultaneously for each applicable factored load combination shall be considered.

410.5 Design Strength

410.5.1 General

410.5.1.1 For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$, including (a) through (d). Interaction between load effects shall be considered:

- a. $\phi P_n \geq P_u$
- b. $\phi M_n \geq M_u$
- c. $\phi V_n \geq V_u$
- d. $\phi T_n \geq T_u$

410.5.1.2 ϕ shall be determined in accordance with Section 421.2.

410.5.2 Axial Force and Moment

410.5.2.1 P_n and M_n shall be calculated in accordance with Section 422.4.

410.5.2.2 For composite columns, forces shall be transferred between the steel section and concrete by direct bearing, shear connectors, or bond in accordance to the axial strength assigned to each component.

410.5.3 Shear

410.5.3.1 V_n shall be calculated in accordance with Section 422.5.

410.5.4 Torsion

410.5.4.1 If $T_u \geq \phi T_{th}$, where T_{th} is given in Section 422.7, torsion shall be considered in accordance with Section 409.

410.6 Reinforcement Limits

410.6.1 Minimum and Maximum Longitudinal Reinforcement

410.6.1.1 For non-prestressed columns and for prestressed columns with average $f_{pe} < 1.6$ MPa, area of longitudinal reinforcement shall be at least $0.01A_g$ but shall not exceed $0.08A_g$.

410.6.1.2 For composite columns with a structural steel core, area of longitudinal bars located within the transverse reinforcement shall be at least $0.01(A_g - A_{sx})$, but shall not exceed $0.08(A_g - A_{sx})$.

410.6.2 Minimum Shear Reinforcement

410.6.2.1 A minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all regions where $V_u > 0.5\phi V_c$.

410.6.2.2 If shear reinforcement is required, $A_{v,min}$ shall be the greater of (a) and (b):

- a. $0.062\sqrt{f'_c} \frac{b_w s}{f_{yt}}$
- b. $0.35 \frac{b_w s}{f_{yt}}$

410.7 Reinforcement Detailing

410.7.1 General

410.7.1.1 Concrete cover for reinforcement shall be in accordance with Section 420.6.1.

410.7.1.2 Development lengths of deformed and prestressed reinforcement shall be in accordance with Section 425.4.

410.7.1.3 Bundled bars shall be in accordance with Section 425.6.

410.7.2 Reinforcement Spacing

410.7.2.1 Minimum spacing s shall be in accordance with Section 425.2.

410.7.3 Longitudinal Reinforcement

410.7.3.1 For non-prestressed columns and for prestressed columns with average $f_{pe} < 1.60$ MPa, the minimum number of longitudinal bars shall be (a), (b), or (c):

- a. Three within triangular ties;
- b. Four within rectangular or circular ties;
- c. Six enclosed by spirals or for columns of special moment frames enclosed by circular hoops.

410.7.3.2 For composite columns with structural steel cores, a longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not farther apart than one-half the least side dimension of the composite column.

410.7.4 Offset Bent Longitudinal Reinforcement

410.7.4.1 The slope of the inclined portion of an offset bent longitudinal bar relative to the longitudinal axis of the column shall not exceed 1 in 6. Portions of bar above and below an offset shall be parallel to axis of column.

410.7.4.2 If the column face is offset 75 mm or more, longitudinal bars shall not be offset bent and separate dowels, lap spliced with the longitudinal bars adjacent to the offset column faces, shall be provided.

410.7.5 Splices of Longitudinal Reinforcement**410.7.5.1 General**

410.7.5.1.1 Lap splices, mechanical splices, butt-welded splices, and end-bearing splices shall be permitted.

410.7.5.1.2 Splices shall satisfy requirements for all factored load combinations.

410.7.5.1.3 Splices of deformed reinforcement shall be in accordance with Section 425.5 and shall satisfy the requirements of Section 410.7.5.2 for lap splices or Section 410.7.5.3 for end-bearing splices.

410.7.5.2 Lap Splices

410.7.5.2.1 If the bar force due to factored loads is compressive, compression lap splices shall be permitted. It shall be permitted to decrease the compression lap splice length in accordance with (a) or (b), but the lap splice length shall be at least 300 mm.:

- a. For tied columns, where ties throughout the lap splice length have an effective area not less than $0.0015h_s$ in both directions, lap splice length shall be permitted to be multiplied by **0.83**. Tie legs perpendicular to dimension h shall be considered in calculating effective area;
- b. For spiral columns, where spirals throughout the lap splice length satisfy Section 425.7.3, lap splice length shall be permitted to be multiplied by 0.75.

410.7.5.2.2 If the bar force due to factored loads is tensile, tension lap splices shall be in accordance with Table 410.7.5.2.2.

Table 410.7.5.2.2 Tension Lap Splice Class

Tensile bar stress	Splice details	Splice type
$\leq 0.5f_y$	$\leq 50\%$ bars spliced at any section and lap splices on adjacent bars staggered by at least ℓ_d	Class A
	Other	Class B
$> 0.5f_y$	All cases	Class B

410.7.5.3 End-bearing Splices

410.7.5.3.1 If the bar force due to factored loads is compressive, end-bearing splices shall be permitted provided the splices are staggered or additional bars are provided at splice locations. The continuing bars in each face of the column shall have a tensile strength at least $0.25f_y$ times the area of the vertical reinforcement along that face.

410.7.5.3.2 For composite columns, ends of structural steel cores shall be accurately finished to bear at end-bearing splices, with positive provision for alignment of one core above the other in concentric contact. Bearing shall be considered effective to transfer not greater than 50 percent of the total compressive force in the steel core.

410.7.6 Transverse Reinforcement**410.7.6.1 General**

410.7.6.1.1 Transverse reinforcement shall satisfy the most restrictive requirements for reinforcement spacing.

410.7.6.1.2 Details of transverse reinforcement shall be in accordance with Section 425.7.2 for ties, Section 425.7.3 for spirals, or Section 425.7.4 for hoops.

410.7.6.1.3 For prestressed columns with average $f_{pe} \geq 1.6$ MPa, transverse ties or hoops need not satisfy the $16d_b$ spacing requirement of Section 425.7.2.1.

410.7.6.1.4 For composite columns with a structural steel core, transverse ties or hoops shall have a minimum d_b of **0.02** times the greater side dimension of the composite column, but shall be at least 10 mm ϕ and need not be larger than 16 mm ϕ . Spacing shall satisfy Section 425.7.2.1, but not exceed **0.5** times the least dimension of the composite column. Deformed wire or welded wire reinforcement of equivalent area shall be permitted.

410.7.6.1.5 Longitudinal reinforcement shall be laterally supported using ties or hoops in accordance with Section 410.7.6.2 or spirals in accordance with Section 410.7.6.3, unless tests and structural analyses demonstrate adequate strength and feasibility of construction.

410.7.6.1.6 If anchor bolts are placed in the top of a column or pedestal, the bolts shall be enclosed by transverse reinforcement that also surrounds at least four longitudinal bars within the column or pedestal. The transverse reinforcement shall be distributed within 125 mm. of the top of the column or pedestal and shall consist of at least two 12 mm ϕ or three 10 mm ϕ bars.

410.7.6.2 Lateral Support of Longitudinal Bars Using Ties or Hoops

410.7.6.2.1 In any storey, the bottom tie or hoop shall be located not more than one-half the tie or hoop spacing above the top of footing or slab.

410.7.6.2.2 In any storey, the top tie or hoop shall be located not more than one-half the tie or hoop spacing below the lowest horizontal reinforcement in the slab, drop panel, or shear cap. If beams or brackets frame into all sides of the column, the top tie or hoop shall be located not more than 75 mm below the lowest horizontal reinforcement in the shallowest beam or bracket.

410.7.6.3 Lateral Support of Longitudinal Bars Using Spirals

410.7.6.3.1 In any storey, the bottom of the spiral shall be located at the top of footing or slab.

410.7.6.3.2 In any storey, the top of the spiral shall be located in accordance with Table 410.7.6.3.2.

Table 410.7.6.3.2 Spiral Extension Requirements at Top of Column

Framing at column end	Extension requirements
Beams or brackets frame into all sides of the column	Extend to the level of the lowest horizontal reinforcement in members supported above.
Beams or brackets do not frame into all sides of the column	Extend to the level of the lowest horizontal reinforcement in members supported above. Additional column ties shall extend above termination of spiral to bottom of slab, drop panel, or shear cap.
Columns with capitals	Extend to the level at which the diameter or width of capital is twice that of the column.

410.7.6.4 Lateral Support of Offset Bent Longitudinal Bars

410.7.6.4.1 Where longitudinal bars are offset, horizontal support shall be provided by ties, hoops, spirals, or parts of the floor construction and shall be designed to resist 1.5 times the horizontal component of the calculated force in the inclined portion of the offset bar.

410.7.6.4.2 If transverse reinforcement is provided to resist forces that result from offset bends, ties, hoops, or spirals shall be placed not more than 150 mm. from points of bend.

410.7.6.5 Shear

410.7.6.5.1 If required, shear reinforcement shall be provided using ties, hoops, or spirals.

410.7.6.5.2 Maximum spacing of shear reinforcement shall be in accordance with Table 410.7.6.5.2.

Table 410.7.6.5.2 Maximum Spacing of Shear Reinforcement

V_s		Maximum s , mm.	
		Non-prestressed column	Prestressed column
$\leq 0.33\sqrt{f'_c}b_wd$	Lesser of:	$\frac{d}{2}$	$\frac{3h}{4}$
		600	
$> 0.33\sqrt{f'_c}b_wd$	Lesser of:	$\frac{d}{4}$	$\frac{3h}{8}$
		300	

SECTION 411 WALLS

411.1 Scope

411.1.1 This section shall apply to the design of non-prestressed and prestressed walls including (a) through (c):

- a. Cast-in-place;
- b. Precast in-plant;
- c. Precast on-site including tilt-up.

411.1.2 Design of special structural walls shall be in accordance with Section 418.

411.1.3 Design of plain concrete walls shall be in accordance with Section 414.

411.1.4 Design of cantilever retaining walls shall be in accordance with Sections 422.2 through 422.4, with minimum horizontal reinforcement in accordance with Section 411.6.

411.2 General

411.2.1 Materials

411.2.1.1 Design properties for concrete shall be selected to be in accordance with Section 419.

411.2.1.2 Design properties for steel reinforcement shall be selected to be in accordance with Section 420.

411.2.1.3 Materials, design, and detailing requirements for embedments in concrete shall be in accordance with Section 420.7.

411.2.2 Connection to Other Members

411.2.2.1 For precast walls, connections shall be designed in accordance with Section 416.2.

411.2.2.2 Connections of walls to foundations shall satisfy Section 416.3.

411.2.3 Load Distribution

411.2.3.1 Unless otherwise demonstrated by an analysis, the horizontal length of wall considered as effective for resisting each concentrated load shall not exceed the lesser of the center-to-center distance between loads, and the bearing width plus four times the wall thickness. Effective horizontal length for bearing shall not extend beyond vertical wall joints unless design provides for transfer of forces across the joints.

411.2.4 Intersecting Elements

411.2.4.1 Walls shall be anchored to intersecting elements, such as floors and roofs; columns, pilasters, buttresses, or intersecting walls; and to footings.

411.3 Design Limits

411.3.1 Minimum Wall Thickness

411.3.1.1 Minimum wall thicknesses shall be in accordance with Table 411.3.1.1. Thinner walls are permitted if adequate strength and stability can be demonstrated by structural analysis

Table 411.3.1.1
Minimum Wall Thickness, h

Wall type	Minimum thickness, h		
Bearing ^[1]	Greater of:	100 mm	a.
		1/25 the lesser of unsupported length and unsupported height	b.
Non-bearing	Greater of:	100 mm	c.
		1/30 the lesser of unsupported length and unsupported height	d.
Exterior basement and Foundation ^[1]		190 mm	e.

^[1] Only applies to walls designed in accordance with the simplified design method of Section 411.5.3.

411.4 Required Strength

411.4.1 General

411.4.1.1 Required strength shall be calculated in accordance with the factored load combinations in Section 405.

411.4.1.2 Required strength shall be calculated in accordance with the analysis procedures in Section 406.

411.4.1.3 Slenderness effects shall be calculated in accordance with Sections 406.6.4, 406.7, or 406.8. Alternatively, out-of-plane slenderness analysis shall be permitted using Section 411.8 for walls meeting the requirements of that section.

411.4.1.4 Walls shall be designed for eccentric axial loads and any lateral or other loads to which they are subjected.

411.4.2 Factored Axial Force and Moment

411.4.2.1 Walls shall be designed for the maximum factored moment, M_u , that can accompany the factored axial force for each applicable load combination. The factored axial force, P_u , at given eccentricity shall not exceed ϕP_n , max where P_n , max shall be as given in Section 422.4.2.1 and strength reduction factor ϕ shall be that for compression-controlled sections in Section 421.2.2. The maximum factored moment M_u shall be magnified for slenderness effects in accordance with Sections 406.6.4, 406.7, or 406.8.

411.4.3 Factored Shear

411.4.3.1 Walls shall be designed for the maximum in-plane V_u and out-of-plane V_u .

411.5 Design Strength

411.5.1 General

411.5.1.1 For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$, including (a) through (c). Interaction between axial load and moment shall be considered.

- a. $\phi P_n \geq P_u$
- b. $\phi M_n \geq M_u$
- c. $\phi V_n \geq V_u$

411.5.1.2 ϕ shall be determined in accordance with Section 421.2.

411.5.2 Axial Load and In-plane or Out-of-plane Flexure

411.5.2.1 For bearing walls, P_n and M_n (in-plane or out-of-plane) shall be calculated in accordance with Section 422.4. Alternatively, axial load and out-of-plane flexure shall be permitted to be considered in accordance with Section 411.5.3.

411.5.2.2 For nonbearing walls, M_n shall be calculated in accordance with Section 422.3.

411.5.3 Axial Load and Out-of-Plane Flexure – Simplified Design Method

411.5.3.1 If the resultant of all factored loads is located within the middle third of the thickness of a solid wall with a rectangular cross section, P_n shall be permitted to be calculated by:

$$\phi P_n = 0.55 \phi f'_c A_g \left[1 - \left(\frac{k \ell_c}{32h} \right)^2 \right] \quad (411.5.3.1)$$

411.5.3.2 Effective length factor, k , for use with Eq. 411.5.3.1 shall be in accordance with Table 411.5.3.2.

Table 411.5.3.2 Effective Length Factor k for Walls

Boundary conditions	k
Walls braced top and bottom against lateral translation and	
a. Restrained against rotation at one or both ends (top, bottom, or both)	0.8
b. Unrestrained against rotation at both ends	1.0
Walls not braced against lateral translation	2.0

411.5.3.3 ϕ in Eq. 411.5.3.1 shall be that for compression controlled sections in Section 421.2.2.

411.5.3.4 Wall reinforcement shall be at least that required by Section 411.6.

411.5.4 In-plane Shear

411.5.4.1 V_n shall be calculated in accordance with Sections 411.5.4.2 through 411.5.4.8. Alternatively, for walls with $h_w \leq 2\ell_w$, it shall be permitted to design for in-plane shear in accordance with the strut-and-tie method of Section 423. In all cases, reinforcement shall satisfy the limits of Sections 411.6, 411.7.2, and 411.7.3.

411.5.4.2 For in-plane shear design, is h thickness of wall and d shall be taken equal to $0.8\ell_w$. A larger value of d , equal to the distance from extreme compression fiber to center of force of all reinforcement in tension, shall be permitted if the center of tension is calculated by a strain compatibility analysis.

411.5.4.3 V_n at any horizontal section shall not exceed $0.83\sqrt{f'_c}hd$.

411.5.4.4 V_n shall be calculated by:

$$V_n = V_c + V_s \quad (411.5.4.4)$$

411.5.4.5 Unless a more detailed calculation is made in accordance with Section 411.5.4.6, V_c shall not exceed $0.17\lambda\sqrt{f'_c}hd$ for walls subject to axial compression or exceed the value given in Section 422.5.7 for walls subject to axial tension.

411.5.4.6 It shall be permitted to calculate V_c in accordance with Table 411.5.4.6, where N_u is positive for compression and negative for tension and the quantity N_u/A_g is expressed in kPa.

411.5.4.7 Sections located closer to wall base than a distance $\ell_w/2$ or one-half the wall height, whichever is less, shall be permitted to be designed for V_c calculated using the detailed calculation options in Table 411.5.4.6 at a distance above the base of $\ell_w/2$ or one-half the wall height, whichever is less.

411.5.4.8 V_s shall be provided by transverse shear reinforcement and shall be calculated by:

$$V_s = \frac{A_v f_{yt} d}{s} \quad (411.5.4.8)$$

411.5.5 Out-of-plane Shear

411.5.5.1 V_n shall be calculated in accordance with Section 422.5.

411.6 Reinforcement Limits

411.6.1 If in-plane $V_u \leq 0.5\phi V_c$, minimum ρ_ℓ and minimum ρ_t shall be in accordance with Table 411.6.1. These limits need not be satisfied if adequate strength and stability can be demonstrated by structural analysis.

411.6.2 If in-plane $V_u \geq 0.5\phi V_c$, (a) and (b) shall be satisfied:

- ρ_ℓ shall be at least the greater of the value calculated by Eq. 411.6.2 and 0.0025, but need not exceed ρ_t in accordance with Table 411.6.1.

$$\rho_\ell \geq 0.0025 + 0.5(2.5 - h_w/\ell_w)(\rho_t - 0.0025) \quad (411.6.2)$$

- ρ_t shall be at least 0.0025

Table 411.5.4.6 V_c : Non-Prestressed and Prestressed Walls

Calculation Option	Axial force	V_c		
Simplified	Compression	$0.17\lambda\sqrt{f'_c}hd$		(a)
	Tension	Greater of:	$0.17\left(1 + \frac{0.29N_u}{A_g}\right)\lambda\sqrt{f'_c}hd$	(b)
			0	(c)
Detailed	Tension or Compression	Lesser of:	$0.27\lambda\sqrt{f'_c}hd + \frac{N_u d}{4\ell_w}$	(d)
			$\left[0.05\lambda\sqrt{f'_c} + \frac{\ell_w\left(0.1\lambda\sqrt{f'_c} + 0.2\frac{N_u}{\ell_w h}\right)}{\frac{M_u}{V_u} - \frac{\ell_w}{2}}\right]hd$ Equation shall not apply if $(M_u/V_u - \ell_w/2)$ is negative.	(e)

Table 411.6.1 Minimum Reinforcement for Walls with In-plane $V_u \leq 0.5\phi V_c$

Wall Type	Type of Non-prestressed Reinforcement	Bar/Wire Size	f_y , MPa	Minimum Longitudinal ^[2] , ρ_ℓ	Minimum Transverse, ρ_t
Cast-in-place	Deformed Bars	$\leq 16 \text{ mm } \phi$	≥ 420	0.0012	0.0020
			< 420	0.0015	0.0025
		$> 16 \text{ mm } \phi$	Any	0.0015	0.0025
	Welded Wire Reinforcement	$\leq \text{MW200 or MD200}$	Any	0.0012	0.0020
Precast ^[1]	Deformed Bars or Welded Wire Reinforcement	Any	Any	0.0010	0.0010

[1] Prestressed walls with an average effective compressive stress of at least 1.6 MPa need not meet the requirement for minimum longitudinal reinforcement, ρ_ℓ .

[2] In one-way precast, prestressed walls not wider than 3.7 m. and not mechanically connected to cause restraint in the transverse direction, the minimum reinforcement requirement in the direction normal to the flexural reinforcement need not be satisfied.

411.7 Reinforcement Detailing

411.7.1 General

411.7.1.1 Concrete cover for reinforcement shall be in accordance with Section 420.6.1.

411.7.1.2 Development lengths of deformed and prestressed reinforcement shall be in accordance with Section 425.4.

411.7.1.3 Splice lengths of deformed reinforcement shall be in accordance with Section 425.5.

411.7.2 Spacing of Longitudinal Reinforcement

411.7.2.1 Spacing, s , of longitudinal bars in cast-in-place walls shall not exceed the lesser of $3h$ and 450 mm. If shear reinforcement is required for in-plane strength, spacing of longitudinal reinforcement shall not exceed $\ell_w/3$.

411.7.2.2 Spacing, s , of longitudinal bars in precast walls shall not exceed the lesser of (a) and (b):

- $5h$;
- 450 mm. for exterior walls or 750 mm. for interior walls.

If shear reinforcement is required for in-plane strength, s shall not exceed the smallest of $3h$, 450 mm., and $\ell_w/3$.

411.7.2.3 For walls with h greater than 250 mm., except basement walls and cantilever retaining walls, distributed reinforcement for each direction shall be placed in two layers parallel with wall faces in accordance with (a) and (b):

- One layer consisting of at least 1/2 and not exceeding 2/3 of total reinforcement required for each direction shall be placed at least 50 mm., but not exceeding $h/3$, from the exterior surface;
- The other layer consisting of the balance of required reinforcement in that direction, shall be placed at least 20 mm, but not greater than $h/3$, from the interior surface.

411.7.2.4 Flexural tension reinforcement shall be well distributed and placed as close as practicable to the tension face.

411.7.3 Spacing of Transverse Reinforcement

411.7.3.1 Spacing, s , of transverse reinforcement in cast-in-place walls shall not exceed the lesser of $3h$ and 450 mm. If shear reinforcement is required for in-plane strength, s shall not exceed $\ell_w/3$.

411.7.3.2 Spacing, s , of transverse bars in precast walls shall not exceed the lesser of (a) and (b):

- $5h$;
- 450 mm. for exterior walls or 750 mm. for interior walls.

If shear reinforcement is required for in-plane strength, s shall not exceed the smallest of $3h$, 450 mm., and $\ell_w/5$.

411.7.4 Lateral Support of Longitudinal Reinforcement

411.7.4.1 If longitudinal reinforcement is required for axial strength or if A_{st} exceeds $0.01A_g$, longitudinal reinforcement shall be laterally supported by transverse ties.

411.7.5 Reinforcement Around Openings

411.7.5.1 In addition to the minimum reinforcement required by Section 411.6, at least two 16 mm ϕ bars in walls having two layers of reinforcement in both directions and one 16 mm ϕ bar in walls having a single layer of reinforcement in both directions shall be provided around window, door, and similarly sized openings. Such bars shall be anchored to develop f_y in tension at the corners of the openings.

411.8 Alternative Method for Out-of-Plane Slender Wall Analysis

411.8.1 General

411.8.1.1 It shall be permitted to analyze out-of-plane slenderness effects in accordance with this section for walls satisfying (a) through (e):

- Cross section is constant over the height of the wall;
- Wall is tension-controlled for out-of-plane moment effect;
- ϕM_n is at least M_{cr} , where M_{cr} is calculated using f_r as provided in Section 419.2.3;
- P_u at the mid-height section does not exceed $0.06f'_cA_g$;
- Calculated out-of-plane deflection due to service loads, Δ_s , including $P\Delta$ effects, does not exceed $\ell_c/150$.

411.8.2 Modeling

411.8.2.1 The wall shall be analyzed as a simply-supported, axially-loaded member subject to an out-of-plane uniformly distributed lateral load, with maximum moments and deflections occurring at mid-height.

411.8.2.2 Concentrated gravity loads applied to the wall above any section shall be assumed to be distributed over a width equal to the bearing width, plus a width on each side that increases at a slope of 2 vertical to 1 horizontal, but not extending beyond (a) or (b):

- The spacing of the concentrated loads;
- The edges of the wall panel.

411.8.3 Factored Moment

411.8.3.1 M_u at mid-height of wall due to combined flexure and axial loads shall include the effects of wall deflection in accordance with (a) or (b):

- By iterative calculation using

$$M_u = M_{ua} + P_u \Delta_u \quad (411.8.3.1a)$$

where M_{ua} is the maximum factored moment at midheight of wall due to lateral and eccentric vertical loads, not including $P\Delta$ effects.

Δ_u shall be calculated by:

$$\Delta_u = \frac{5M_u \ell_c^2}{(0.75)48E_c I_{cr}} \quad (411.8.3.1b)$$

where I_{cr} shall be calculated by:

$$I_{cr} = \frac{E_s}{E_c} \left(A_s + \frac{P_u h}{f_y 2d} \right) (d - c)^2 + \frac{\ell_w c^3}{3} \quad (411.8.3.1c)$$

and the value of E_s/E_c shall be at least 6.

- By direct calculation using:

$$M_u = \frac{M_{ua}}{\left(1 - \frac{5P_u \ell_c}{(0.75)48E_c I_{cr}} \right)} \quad (411.8.3.1d)$$

411.8.4 Out-of-Plane Deflection – Service Loads

411.8.4.1 Out-of-plane deflection due to service loads, Δ_s , shall be calculated in accordance with Table 411.8.4.1, where M_a is calculated by Section 411.8.4.2.

Table 411.8.4.1 Calculation of Δ_s

M_a	Δ_s	
$\leq (2/3)M_{cr}$	$\Delta_s = \left(\frac{M_a}{M_{cr}}\right) \Delta_{cr}$	(a)
$> (2/3)M_{cr}$	$\Delta_s = (2/3)\Delta_{cr} + \frac{(M_a - (2/3)M_{cr})}{(M_n - (2/3)M_{cr})}$	(b)

411.8.4.2 The maximum moment M_a at mid-height of wall due to service lateral and eccentric vertical loads, including $P_s\Delta_s$ effects, shall be calculated by Eq. 411.8.4.2 with iteration of deflections.

$$M_a = M_{sa} + P_s\Delta_s \quad (411.8.4.2)$$

411.8.4.3 Δ_{cr} and Δ_n shall be calculated by (a) and (b):

$$a. \quad \Delta_{cr} = \frac{5M_{cr}\ell_c^2}{48E_cI_g} \quad (411.8.4.3a)$$

$$b. \quad \Delta_n = \frac{5M_n\ell_c^2}{48E_cI_{cr}} \quad (411.8.4.3b)$$

411.8.4.4 I_{cr} shall be calculated by Eq. 411.8.3.1c.

SECTION 412 DIAPHRAGMS

412.1 Scope

412.1.1 This section shall apply to the design of non-prestressed and prestressed diaphragms, including (a) through (d):

- Diaphragms that are cast-in-place slabs;
- Diaphragms that comprise a cast-in-place topping slab on precast elements;
- Diaphragms that comprise precast elements with end strips formed by either a cast-in-place concrete topping slab or edge beams;
- Diaphragms of interconnected precast elements without cast-in-place concrete topping.

412.1.2 Diaphragms in structures assigned to seismic zone 4 shall also satisfy requirements of Section 418.12.

412.2 General

412.2.1 Design shall consider (a) through (e):

- Diaphragm in-plane forces due to lateral loads acting on the building;
- Diaphragm transfer forces;
- Connection forces between the diaphragm and vertical framing or nonstructural elements;
- Forces resulting from bracing vertical or sloped building elements;
- Diaphragm out-of-plane forces due to gravity and other loads applied to the diaphragm surface.

412.2.2 Materials

412.2.2.1 Design properties for concrete shall be selected to be in accordance with Section 419.

412.2.2.2 Design properties for steel reinforcement shall be selected to be in accordance with Section 420.

412.3 Design Limits

412.3.1 Minimum Diaphragm Thickness

412.3.1.1 Diaphragms shall have thickness as required for stability, strength, and stiffness under factored load combinations.

412.3.1.2 Floor and roof diaphragms shall have a thickness not less than that required for floor and roof elements in other parts of this Code.

412.4 Required Strength

412.4.1 General

412.4.1.1 Required strength of diaphragms, collectors, and their connections shall be calculated in accordance with the factored load combinations in Section 405.

412.4.1.2 Required strength of diaphragms that are part of floor or roof construction shall include effects of out-of-plane loads simultaneous with other applicable loads.

412.4.2 Diaphragm Modeling and Analysis

412.4.2.1 Diaphragm modeling and analysis requirements of the general building code shall govern where applicable. Otherwise, diaphragm modeling and analysis shall be in accordance with Sections 412.4.2.2 through 412.4.2.4.

412.4.2.2 Modeling and analysis procedures shall satisfy requirements of Section 406.

412.4.2.3 Any set of reasonable and consistent assumptions for diaphragm stiffness shall be permitted.

412.4.2.4 Calculation of diaphragm in-plane design moments, shears, and axial forces shall be consistent with requirements of equilibrium and with design boundary conditions. It shall be permitted to calculate design moments, shears, and axial forces in accordance with one of (a) through (e):

- a. A rigid diaphragm model if the diaphragm can be idealized as rigid;
- b. A flexible diaphragm model if the diaphragm can be idealized as flexible;
- c. A bounding analysis in which the design values are the envelope of values obtained by assuming upper bound and lower bound in-plane stiffnesses for the diaphragm in two or more separate analyses;

d. A finite element model considering diaphragm flexibility;

e. A strut-and-tie model in accordance with Section 423.2.

412.5 Design Strength

412.5.1 General

412.5.1.1 For each applicable factored load combination, design strengths of diaphragms and connections shall satisfy $\phi S_n \geq U$. Interaction between load effects shall be considered.

412.5.1.2 ϕ shall be determined in accordance with Section 421.2.

412.5.1.3 Design strengths shall be in accordance with (a), (b), (c), or (d):

- a. For a diaphragm idealized as a beam whose depth is equal to the full diaphragm depth, with moment resisted by boundary reinforcement concentrated at the diaphragm edges, design strengths shall be in accordance with Sections 412.5.2 through 412.5.4;
- b. For a diaphragm or a diaphragm segment modeled as a strut-and-tie system, design strengths shall be in accordance with Section 423.3;
- c. For a diaphragm idealized with a finite-element model, design strengths shall be in accordance with Section 422. Non-uniform shear distributions shall be considered in design for shear. Collectors in such designs shall be provided to transfer diaphragm shears to the vertical elements of the lateral-force-resisting system;
- d. For a diaphragm designed by alternative methods, such methods shall satisfy the requirements of equilibrium and shall provide design strengths at least equal to required strengths for all elements in the load path.

412.5.1.4 It shall be permitted to use pre-compression from prestressed reinforcement to resist diaphragm forces.

412.5.1.5 If non-prestressed, bonded prestressing reinforcement is designed to resist collector forces, diaphragm shear, or tension due to in-plane moment, the value of steel stress used to calculate resistance shall not exceed the lesser of the specified yield strength and 420 MPa.

412.5.2 Moment and Axial Force

412.5.2.1 It shall be permitted to design a diaphragm to resist in-plane moment and axial force in accordance with Sections 422.3 and 422.4.

412.5.2.2 It shall be permitted to resist tension due to moment by (a), (b), (c), or (d), or those methods in combination:

- a. Deformed bars conforming to Section 420.2.1;
- b. Strands or bars conforming to Section 420.3.1, either prestressed or non-prestressed;
- c. Mechanical connectors crossing joints between precast elements;
- d. Pre-compression from prestressed reinforcement.

412.5.2.3 Non-prestressed reinforcement and mechanical connectors resisting tension due to moment shall be located within $h/4$ of the tension edge of the diaphragm, where h is diaphragm depth measured in the plane of the diaphragm at that location. Where diaphragm depth changes along the span, it shall be permitted to develop reinforcement into adjacent diaphragm segments that are not within the $h/4$ limit.

412.5.2.4 Mechanical connectors crossing joints between precast elements shall be designed to resist required tension under the anticipated joint opening.

412.5.3 Shear

412.5.3.1 This section shall apply to diaphragm in-plane shear strength.

412.5.3.2 ϕ shall be 0.75, unless a lesser value is required by Section 421.2.4.

412.5.3.3 For a diaphragm that is entirely cast-in-place, V_n shall be calculated by Eq. 412.5.3.3.

$$V_n = A_{cv}(0.17\lambda\sqrt{f'_c} + \rho_t f_y) \quad (412.5.3.3)$$

where A_{cv} is the gross area of concrete bounded by diaphragm web thickness and depth, reduced by void areas if present; the value of $\sqrt{f'_c}$ used to calculate V_n shall not exceed 8.3 MPa; and ρ_t is distributed reinforcement oriented parallel to the in-plane shear.

412.5.3.4 For a diaphragm that is entirely cast-in-place, cross-sectional dimensions shall be selected to satisfy Eq. 412.5.3.4.

$$V_u \leq 0.66\phi A_{cv}\sqrt{f'_c} \quad (412.5.3.4)$$

where the value of $\sqrt{f'_c}$ used to calculate V_n shall not exceed 8.3 MPa.

412.5.3.5 For diaphragms that are cast-in-place concrete topping slabs on precast elements, (a) and (b) shall be satisfied:

- a. V_n shall be calculated in accordance with Eq. 412.5.3.3, and cross-sectional dimensions shall be selected to satisfy Eq. 412.5.3.4. A_{cv} shall be calculated using the thickness of the topping slab for non-composite topping slab diaphragms and the combined thickness of cast-in-place and precast elements for composite topping slab diaphragms. For composite topping slab diaphragms, the value of f'_c in Eqs. 412.5.3.3 and 412.5.3.4 shall not exceed the lesser of f'_c for the precast members and f'_c for the topping slab;
- b. V_n shall not exceed the value calculated in accordance with the shear-friction provisions of Section 422.9 considering the thickness of the topping slab above joints between precast elements in non-composite and composite topping slab diaphragms and the reinforcement crossing the joints between the precast members.

412.5.3.6 For diaphragms that are interconnected precast elements without a concrete topping, and for diaphragms that are precast elements with end strips formed by either a cast-in-place concrete topping slab or edge beams, it shall be permitted to design for shear in accordance with (a), (b), or both.

- a. The nominal strength of grouted joints shall not exceed 0.55 MPa. Reinforcement shall be designed to resist shear through shear-friction in accordance with Section 422.9. Shear-friction reinforcement shall be in addition to reinforcement designed to resist tension due to moment and axial force;
- b. Mechanical connectors crossing joints between precast elements shall be designed to resist required shear under anticipated joint opening.

412.5.3.7 For any diaphragm, where shear is transferred from the diaphragm to a collector, or from the diaphragm or collector to a vertical element of the lateral-force-resisting system, (a) or (b) shall apply:

- a. Where shear is transferred through concrete, the shear-friction provisions of Section 422.9 shall be satisfied;
- b. Where shear is transferred through mechanical connectors or dowels, effects of uplift and rotation of the vertical element of the lateral-force-resisting system shall be considered.

412.5.4 Collectors

412.5.4.1 Collectors shall extend from the vertical elements of the lateral-force-resisting system across all or part of the diaphragm depth as required to transfer shear from the diaphragm to the vertical element. It shall be permitted to discontinue a collector along lengths of vertical elements of the lateral force-resisting system where transfer of design collector forces is not required.

412.5.4.2 Collectors shall be designed as tension members, compression members, or both, in accordance with Section 422.4.

412.5.4.3 Where a collector is designed to transfer forces to a vertical element, collector reinforcement shall extend along the vertical element at least the greater of (a) and (b):

- a. The length required to develop the reinforcement in tension;
- b. The length required to transmit the design forces to the vertical element through shear-friction in accordance with Section 422.9, through mechanical connectors, or through other force transfer mechanisms.

412.6 Reinforcement Limits

412.6.1 Reinforcement to resist shrinkage and temperature stresses shall be in accordance with Section 424.4.

412.6.2 Except for slabs-on-ground, diaphragms that are part of floor or roof construction shall satisfy reinforcement limits for one-way slabs in accordance with Section 407.6 or two-way slabs in accordance with Section 408.6, as applicable.

412.6.3 Reinforcement designed to resist diaphragm in-plane forces shall be in addition to reinforcement designed to resist other load effects, except reinforcement designed to resist shrinkage and temperature effects shall be permitted to also resist diaphragm in-plane forces

412.7 Reinforcement Detailing

412.7.1 General

412.7.1.1 Concrete cover for reinforcement shall be in accordance with Section 420.6.1.

412.7.1.2 Development lengths of deformed and prestressed reinforcement shall be in accordance with Section 425.4, unless longer lengths are required by Section 418.

412.7.1.3 Splices of deformed reinforcement shall be in accordance with Section 425.5.

412.7.1.4 Bundled bars shall be in accordance with Section 425.6.

412.7.2 Reinforcement Spacing

412.7.2.1 Minimum spacing s of reinforcement shall be in accordance with Section 425.2.

412.7.2.2 Maximum spacing s of deformed reinforcement shall be the lesser of five times the diaphragm thickness and 450 mm.

412.7.3 Diaphragm and Collector Reinforcement

412.7.3.1 Except for slabs-on-ground, diaphragms that are part of floor or roof construction shall satisfy reinforcement detailing of one-way slabs in accordance with Section 407.7 or two-way slabs in accordance with Section 408.7, as applicable.

412.7.3.2 Calculated tensile or compressive force in reinforcement at each section of the diaphragm or collector shall be developed on each side of that section.

412.7.3.3 Reinforcement provided to resist tension shall extend beyond the point at which it is no longer required to resist tension at least ℓ_d , except at diaphragm edges and at expansion joints.

SECTION 413 FOUNDATIONS

413.1 Scope

413.1.1 This section shall apply to the design of non-prestressed and prestressed foundations, including shallow foundations (a) through (e) and, where applicable, deep foundations (f) through (i):

- a. Strip footings;
- b. Isolated footings;
- c. Combined footings;
- d. Mat foundations;
- e. Grade beams;
- f. Pile caps;
- g. Piles;
- h. Drilled piers;
- i. Caissons.

413.1.2 Foundations excluded by Section 401.4.6 are excluded from this section.

413.2 General

413.2.1 Materials

413.2.1.1 Design properties for concrete shall be selected to be in accordance with Section 419.

413.2.1.2 Design properties for steel reinforcement shall be selected to be in accordance with Section 420.

413.2.1.3 Materials, design, and detailing requirements for embedments in concrete shall be in accordance with Section 420.7.

413.2.2 Connection to Other Members

413.2.2.1 Design and detailing of cast-in-place and precast column, pedestal, and wall connections to foundations shall be in accordance with Section 416.3.

413.2.3 Earthquake Effects

413.2.3.1 Structural members extending below the base of the structure that are required to transmit forces resulting from earthquake effects to the foundation shall be designed in accordance with Section 418.2.2.3.

413.2.3.2 For structures assigned to seismic zone 4, shallow and deep foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground shall be designed in accordance with Section 418.13.

413.2.4 Slabs-On-Ground

413.2.4.1 Slabs-on-ground that transmit vertical loads or lateral forces from other parts of the structure to the ground shall be designed and detailed in accordance with applicable provisions of this Code.

413.2.4.2 Slabs-on-ground that transmits lateral forces as part of the seismic-force-resisting system shall be designed in accordance with Section 418.13.

413.2.5 Plain Concrete

413.2.5.1 Plain concrete foundations shall be designed in accordance with Section 414.

413.2.6 Design Criteria

413.2.6.1 Foundations shall be proportioned to resist factored loads and induced reactions.

413.2.6.2 Foundation systems shall be permitted to be designed by any procedure satisfying equilibrium and geometric compatibility.

413.2.6.3 Foundation design in accordance with strut-and-tie modeling, Section 423, shall be permitted.

413.2.6.4 External moment on any section of a strip footing, isolated footing, or pile cap shall be calculated by passing a vertical plane through the member and calculating the moment of the forces acting over the entire area of member on one side of that vertical plane.

413.2.7 Critical Sections for Shallow Foundations and Pile Caps

413.2.7.1 M_u at the supported member shall be permitted to be calculated at the critical section defined in accordance with Table 413.2.7.1.

Table 413.2.7.1
Location of Critical Section for M_u

Supported member	Location of critical section
Column or pedestal	Face of column or pedestal
Column with steel base plate	Halfway between face of column and edge of steel base plate
Concrete wall	Face of wall
Masonry wall	Halfway between center and face of masonry wall

413.2.7.2 The location of critical section for factored shear in accordance with Sections 407.4.3 and 408.4.3 for one-way shear or Section 408.4.4.1 for two-way shear shall be measured from the location of the critical section for M_u in Section 413.2.7.1.

413.2.7.3 Circular or regular polygon-shaped concrete columns or pedestals shall be permitted to be treated as square members of equivalent area when locating critical sections for moment, shear, and development of reinforcement.

413.2.8 Development of Reinforcement in Shallow Foundations and Pile Caps

413.2.8.1 Development of reinforcement shall be in accordance with Section 425.

413.2.8.2 Calculated tensile or compression force in reinforcement at each section shall be developed on each side of that section.

413.2.8.3 Critical sections for development of reinforcement shall be assumed at the same locations as given in Section 413.2.7.1 for maximum factored moment and at all other vertical planes where changes of section or reinforcement occur.

413.2.8.4 Adequate anchorage shall be provided for tension reinforcement where reinforcement stress is not directly proportional to moment, such as in sloped, stepped, or tapered foundations; or where tension reinforcement is not parallel to the compression face.

413.3 Shallow Foundations

413.3.1 General

413.3.1.1 Minimum base area of foundation shall be calculated from unfactored forces and moments transmitted by foundation to soil or rock and permissible bearing pressure selected through principles of soil or rock mechanics.

413.3.1.2 Overall depth of foundation shall be selected such that the effective depth of bottom reinforcement is at least 150 mm.

413.3.1.3 In sloped, stepped, or tapered foundations, depth and location of steps or angle of slope shall be such that design requirements are satisfied at every section.

413.3.2 One-Way Shallow Foundations

413.3.2.1 The design and detailing of one-way shallow foundations, including strip footings, combined footings, and grade beams, shall be in accordance with this section and the applicable provisions of Sections 407 and 409.

413.3.2.2 Reinforcement shall be distributed uniformly across entire width of one-way footings.

413.3.3 Two-Way Isolated Footings

413.3.3.1 The design and detailing of two-way isolated footings shall be in accordance with this section and the applicable provisions of Sections 407 and 408.

413.3.3.2 In square two-way footings, reinforcement shall be distributed uniformly across entire width of footing in both directions.

413.3.3.3 In rectangular footings, reinforcement shall be distributed in accordance with (a) and (b).

- Reinforcement in the long direction shall be distributed uniformly across entire width of footing.
- For reinforcement in the short direction, a portion of the total reinforcement, $\gamma_s A_s$, shall be distributed uniformly over a band width equal to the length of short side of footing, centered on centerline of column or pedestal. Remainder of reinforcement required in the short direction, $(1 - \gamma_s) A_s$, shall be distributed uniformly outside the center band width of footing, where γ_s is calculated by:

$$\gamma_s = \frac{2}{(\beta + 1)} \quad (413.3.3.3)$$

where β is the ratio of long to short side of footing.

413.3.4 Two-Way Combined Footings and Mat Foundations

413.3.4.1 The design and detailing of combined footings and mat foundations shall be in accordance with this section and the applicable provisions of Section 408.

413.3.4.2 The direct design method of Section 408.10 shall not be used to design combined footings and mat foundations.

413.3.4.3 Distribution of bearing pressure under combined footings and mat foundations shall be consistent with properties of the soil or rock and the structure, and with established principles of soil or rock mechanics.

413.3.4.4 Minimum reinforcement in non-prestressed mat foundations shall be in accordance with Section 408.6.1.1.

413.4 Deep Foundations

413.4.1 General

413.4.1.1 Number and arrangement of piles, drilled piers, and caissons shall be determined from unfactored forces and moments transmitted to these members and permissible member capacity selected through principles of soil or rock mechanics.

413.4.2 Pile Caps

413.4.2.1 Overall depth of pile cap shall be selected such that the effective depth of bottom reinforcement is at least 300 mm.

413.4.2.2 Factored moments and shears shall be permitted to be calculated with the reaction from any pile assumed to be concentrated at the centroid of the pile section.

413.4.2.3 Except for pile caps designed in accordance with Section 413.2.6.3, the pile cap shall be designed such that (a) is satisfied for one-way foundations and (a) and (b) are satisfied for two way foundations.

- a. $\phi V_n \geq V_u$, where V_n shall be calculated in accordance with Section 422.5 for one-way shear, V_u shall be calculated in accordance with Section 413.4.2.5, and ϕ shall be in accordance with Section 421.2;
- b. $\phi v_n \geq v_u$, where v_n shall be calculated in accordance with Section 422.6 for two-way shear, v_u shall be calculated in accordance with Section 413.4.2.5, and ϕ shall be in accordance with Section 421.2.

413.4.2.4 If the pile cap is designed in accordance with strut-and-tie modeling as permitted in Section 413.2.6.3, the effective concrete compressive strength of the struts, f_{ce} , shall be calculated in accordance with Section

423.4.3, where $\beta_s = 0.60\lambda$ and λ is in accordance with Section 419.2.4.

413.4.2.5 Calculation of factored shear on any section through a pile cap shall be in accordance with (a) through (c):

- a. Entire reaction from any pile with its center located $d_{pile}/2$ or more outside the section shall be considered as producing shear on that section;
- b. Reaction from any pile with its center located $d_{pile}/2$ or more inside the section shall be considered as producing no shear on that section;
- c. For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the section shall be based on a linear interpolation between full value at $d_{pile}/2$ outside the section and zero value at $d_{pile}/2$ inside the section.

413.4.3 Deep Foundation Members

413.4.3.1 Portions of deep foundation members in air, water, or soils not capable of providing adequate restraint throughout the member length to prevent lateral buckling shall be designed as columns in accordance with the applicable provisions of Section 410.

SECTION 414 PLAIN CONCRETE

414.1 Scope

414.1.1 This section shall apply to the design of plain concrete members, including (a) and (b):

- a. Members in building structures;
- b. Members in non-building structures such as arches, underground utility structures, gravity walls, and shielding walls.

414.1.2 This section shall not govern the design of cast-in-place concrete piles and piers embedded in ground.

414.1.3 Plain concrete shall be permitted only in cases (a) through (d):

- a. Members that are continuously supported by soil or supported by other structural members capable of providing continuous vertical support;
- b. Members for which arch action provides compression under all conditions of loading;
- c. Walls;
- d. Pedestals.

414.1.4 Plain concrete shall be permitted for a structure assigned to seismic zone 4, only in cases (a) and (b):

- a. Footings supporting cast-in-place reinforced concrete or reinforced masonry walls provided the footings are reinforced longitudinally with at least two continuous reinforcing bars. Bars shall be at least 12 mm ϕ and have a total area of not less than 0.002 times the gross cross-sectional area of the footing. Continuity of reinforcement shall be provided at corners and intersections;
- b. Foundation elements (i) through (iii) for detached one- and two-family dwellings not exceeding three storeys and constructed with stud bearing walls:
 - i. Footings supporting walls;
 - ii. Isolated footings supporting columns or pedestals;
 - iii. Foundation or basement walls not less than 190 mm. thick and retaining no more than 1.2 m. of unbalanced fill.

414.1.5 Plain concrete shall not be permitted for columns and pile caps.

414.2 General

414.2.1 Materials

414.2.1.1 Design properties for concrete shall be selected to be in accordance with Section 419.

414.2.1.2 Steel reinforcement, if required, shall be selected to be in accordance with Section 420.

414.2.1.3 Materials, design, and detailing requirements for embedments in concrete shall be in accordance with Section 420.7.

414.2.1 Connection to Other Members

414.2.2.1 Tension shall not be transmitted through outside edges, construction joints, contraction joints, or isolation joints of an individual plain concrete element.

414.2.2.2 Walls shall be braced against lateral translation.

414.2.3 Precast

414.2.3.1 Design of precast members shall consider all loading conditions from initial fabrication to completion of the structure, including form removal, storage, transportation, and erection.

414.2.3.2 Precast members shall be connected to transfer lateral forces into a structural system capable of resisting such forces.

414.3 Design Limits

414.3.1 Bearing Walls

414.3.1.1 Minimum bearing wall thickness shall be in accordance with Table 414.3.1.1.

Table 414.3.1.1 Minimum Thickness of Bearing Walls

Wall type	Minimum thickness	
General	Greater of:	140 mm
		1/24 the lesser of unsupported length and unsupported height
Exterior basement	190 mm	
Foundation	190 mm	

414.3.2 Footings

414.3.2.1 Footing thickness shall be at least 200 mm.

414.3.2.2 Base area of footing shall be determined from unfactored forces and moments transmitted by footing to soil and permissible soil pressure selected through principles of soil mechanics.

414.3.3 Pedestals

414.3.3.1 Ratio of unsupported height to average least lateral dimension shall not exceed 3.

414.3.4 Contraction and Isolation Joints

414.3.4.1 Contraction or isolation joints shall be provided to divide structural plain concrete members into flexurally discontinuous elements. The size of each element shall be selected to limit stress caused by restraint to movements from creep, shrinkage, and temperature effects.

414.3.4.2 The number and location of contraction or isolation joints shall be determined considering (a) through (f):

- Influence of climatic conditions;
- Selection and proportioning of materials;
- Mixing, placing, and curing of concrete;
- Degree of restraint to movement;
- Stresses due to loads to which an element is subjected;
- Construction techniques.

414.4 Required Strength**414.4.1 General**

414.4.1.1 Required strength shall be calculated in accordance with the factored load combinations defined in Section 405.

414.4.1.2 Required strength shall be calculated in accordance with the analysis procedures in Section 406.

414.4.1.3 No flexural continuity due to tension shall be assumed between adjacent structural plain concrete elements.

414.4.2 Walls

414.4.2.1 Walls shall be designed for an eccentricity corresponding to the maximum moment that can accompany the axial load but not less than $0.10h$, where h is the wall thickness.

414.4.3 Footings**414.4.3.1 General**

414.4.3.1.1 For footings supporting circular or regular polygon-shaped concrete columns or pedestals, it shall be permitted to assume a square section of equivalent area for determining critical sections.

414.4.3.2 Factored Moment

414.4.3.2.1 The critical section for M_u shall be located in accordance with Table 414.4.3.2.1.

Table 414.4.3.2.1 Location of Critical Section for M_u

Supported member	Location of critical section
Column or pedestal	Face of column or pedestal
Column with steel base plate	Halfway between face of column and edge of steel base plate
Concrete wall	Face of wall
Masonry wall	Halfway between center and face of masonry wall

414.4.3.3 Factored One-Way Shear

414.4.3.3.1 For one-way shear, critical sections shall be located h from (a) and (b) where h is the footing thickness.

- a. Location defined in Table 414.4.3.2.1;
- b. Face of concentrated loads or reaction areas.

414.4.3.3.2 Sections between (a) or (b) of Section 414.4.3.3.1 and the critical section for shear shall be permitted to be designed for V_u at the critical section for shear.

414.4.3.4 Factored Two-Way Shear

414.4.3.4.1 For two-way shear, critical sections shall be located so that the perimeter b_o is a minimum but need not be closer than $h/2$ to (a) through (c):

- a. Location defined in Table 414.4.3.2.1;
- b. Face of concentrated loads or reaction areas;
- c. Changes in footing thickness.

414.4.3.4.2 For square or rectangular columns, concentrated loads, or reaction areas, critical section for two-way shear shall be permitted to be calculated assuming straight sides.

414.5 Design Strength**414.5.1 General**

414.5.1.1 For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$, including (a) through (d). Interaction between load effects shall be considered.

- a. $\phi M_n \geq M_u$
- b. $\phi P_n \geq P_u$
- c. $\phi V_n \geq V_u$
- d. $\phi B_n \geq B_u$

414.5.1.2 ϕ shall be determined in accordance with Section 421.2.

414.5.1.3 Tensile strength of concrete shall be permitted to be considered in design.

414.5.1.4 Flexure and axial strength calculations shall be based on a linear stress-strain relationship in both tension and compression.

414.5.1.5 λ for lightweight concrete shall be in accordance with Section 419.2.4.

414.5.1.6 No strength shall be assigned to steel reinforcement.

414.5.1.7 When calculating member strength in flexure, combined flexure and axial load, or shear, the entire cross section shall be considered in design, except for concrete cast against soil where overall thickness h shall be taken as 50 mm less than the specified thickness.

414.5.1.8 Unless demonstrated by analysis, horizontal length of wall to be considered effective for resisting each vertical concentrated load shall not exceed center-to-center distance between loads, or bearing width plus four times the wall thickness.

414.5.2 Flexure

414.5.2.1 M_n shall be the lesser of Eq. 414.5.2.1a calculated at the tension face and Eq. 414.5.2.1b calculated at the compression face:

$$M_n = 0.42\lambda\sqrt{f'_c}S_m \quad (414.5.2.1a)$$

$$M_n = 0.85f'_cS_m \quad (414.5.2.1b)$$

where S_m is the corresponding elastic section modulus.

414.5.3 Axial Compression

414.5.3.1 P_n shall be calculated by:

$$P_n = 0.60f'_cA_g \left[1 - \left(\frac{e_c}{32h} \right)^2 \right] \quad (414.5.3.1)$$

414.5.4 Flexure and Axial Compression

414.5.4.1 Unless permitted by Section 414.5.4.2, member dimensions shall be proportioned to be in accordance with Table 414.5.4.1, where M_n is calculated in accordance with Section 414.5.2.1(b) and P_n is calculated in accordance with Section 414.5.3.1.

Table 414.5.4.1
Combined Flexure and Axial Compression

Location	Interaction Equation
Tension face	$\frac{M_u}{S_m} - \frac{P_u}{A_g} \leq 0.42\lambda\sqrt{f'_c}$ (a)
Compression face	$\frac{M_u}{\phi M_n} - \frac{P_u}{\phi P_n} \leq 1.0$ (b)

414.5.4.2 For walls of solid rectangular cross section where $M_u \leq P_u(h/6)$, M_u need not be considered in design and P_n is calculated by

$$P_n = 0.45f'_cA_g \left[1 - \left(\frac{\ell_c}{32h} \right)^2 \right] \quad (414.5.4.2)$$

414.5.5 Shear

414.5.5.1 V_n shall be calculated in accordance with Table 414.5.5.1.

Table 414.5.5.1 Nominal Shear Strength

Shear Action	Nominal shear strength, V_n	
One-way	$0.11\lambda\sqrt{f'_c}b_w h$ (a)	
Two-way	Lesser of:	$\left(1 + \frac{2}{\beta}\right)(0.11\lambda\sqrt{f'_c}b_o h)^{[1]}$ (b)
		$2(0.11\lambda\sqrt{f'_c}b_o h)$ (c)

^[1] β is the ratio of long side to short side of concentrated load or reaction area.

414.5.6 Bearing

414.5.6.1 B_n shall be calculated in accordance with Table 414.5.6.1.

Table 414.5.6.1
Nominal Bearing Strength

Relative Geometric Conditions	B_n	
Supporting surface is wider on all sides than the loaded area	Lesser of:	$\sqrt{A_2/A_1}(0.85f'_cA_1)$
		$2(0.85f'_cA_1)$
Other	$0.85f'_cA_1$	

414.6 Reinforcement Detailing

414.6.1 At least two 16 mm ϕ bars shall be provided around all window and door openings. Such bars shall extend at least 600 mm beyond the corners of openings.

SECTION 415

BEAM-CONCRETE AND SLAB-COLUMN JOINTS

415.1 Scope

415.1.1 This section shall apply to the design and detailing of cast-in-place beam-column and slab-column joints.

415.2 General

415.2.1 Beam-column and slab-column joints shall satisfy Section 415.3 for transfer of column axial force through the floor system.

415.2.2 If gravity load, wind, earthquake, or other lateral forces cause transfer of moment at beam-column or slab-column joints, the shear resulting from moment transfer shall be considered in the design of the joint.

415.2.3 Beam-column and slab-column joints that transfer moment to columns shall satisfy the detailing provisions in Section 415.4. Beam-column joints within special moment frames, slab-column joints within intermediate moment frames, and beam-column and slab-column joints using frames not designated as part of the seismic-force-resisting systems in structures assigned to seismic zone 4, shall satisfy Section 418.

415.2.4 A beam-column joint shall be considered to be restrained if the joint is laterally supported on four sides by beams of approximately equal depth.

415.2.5 A slab-column joint shall be considered to be restrained if the joint is laterally supported on four sides by the slab.

415.3 Transfer of Column Axial Force through the Floor System

415.3.1 If f'_c of a column is greater than 1.4 times that of the floor system, transmission of axial force through the floor system shall in accordance with (a), (b), or (c):

- a. Concrete of compressive strength specified for the column shall be placed in the floor at the column location. Column concrete shall extend outward at least 600 mm. into the floor slab from face of column for the full depth of the slab and be integrated with floor concrete;
- b. The Design strength of a column through a floor system shall be calculated using the lower value of

concrete strength with vertical dowels and spirals as required to achieve adequate strength;

- c. For beam-column and slab-column joints that are restrained in accordance with Section 415.2.4 or 415.2.5, respectively, it shall be permitted to calculate the design strength of the column on an assumed concrete strength in the column joint equal to 75 percent of column concrete strength plus 35 percent of floor concrete strength, where the value of column concrete strength shall not exceed 2.5 times the floor concrete strength.

415.4 Detailing of Joints

415.4.1 Beam-column and slab-column joints that are restrained in accordance with Section 415.2.4 or 415.2.5, respectively, and are not part of a seismic-force-resisting system need not satisfy the provisions for transverse reinforcement of Section 415.4.2.

415.4.2 The area of all legs of transverse reinforcement in each principal direction of beam-column and slab-column joints shall be at least the greater of (a) and (b):

- a. $0.062\sqrt{f'_c}\frac{bs}{f_{yt}}$
- b. $0.35\frac{bs}{f_{yt}}$

where b is the dimension of the column section perpendicular to the direction under consideration.

415.4.2.1 At beam-column and slab-column joints, an area of transverse reinforcement calculated in accordance with Section 415.4.2 shall be distributed within the column height not less than the deepest beam or slab element framing into the column.

415.4.2.2 For beam-column joints, the spacing of the transverse reinforcement s shall not exceed one-half the depth of the shallowest beam.

415.4.3 If longitudinal beam or column reinforcement is spliced or terminated in a joint, closed transverse reinforcement in accordance with Section 410.7.6 shall be provided in the joint, unless the joint region is restrained in accordance with Section 415.2.4 or 415.2.5.

415.4.4 Development of longitudinal reinforcement terminating in the joint shall be in accordance with Section 425.4.

SECTION 416

CONNECTION BETWEEN MEMBERS

416.1 Scope

416.1.1 This section shall apply to the design of joints and connections at the intersection of concrete members and for load transfer between concrete surfaces, including (a) through (d):

- Connections of precast members;
- Connections between foundations and either cast-in-place or precast members;
- Horizontal shear strength of composite concrete flexural members;
- Brackets and corbels.

416.2 Connections of Precast Members

416.2.1 General

416.2.1.1 Transfer of forces by means of grouted joints, shear keys, bearing, anchors, mechanical connectors, steel reinforcement, reinforced topping, or a combination of these, shall be permitted.

416.2.1.2 Adequacy of connections shall be verified by analysis or test.

416.2.1.3 Connection details that rely solely on friction caused by gravity loads shall not be permitted.

416.2.1.4 Connections, and regions of members adjacent to connections, shall be designed to resist forces and accommodate deformations due to all load effects in the precast structural system.

416.2.1.5 Design of connections shall consider structural effects of restraint of volume change in accordance with Section 405.3.6.

416.2.1.6 Design of connections shall consider the effects of tolerances specified for fabrication and erection of precast members.

416.2.1.7 Design of a connection with multiple components shall consider the differences in stiffness, strength, and ductility of the components.

416.2.1.8 Integrity ties shall be provided in the vertical, longitudinal, and transverse directions and around the perimeter of a structure in accordance with Section 416.2.4 or 416.2.5.

416.2.2 Required Strength

416.2.2.1 Required strength of connections and adjacent regions shall be calculated in accordance with the factored load combinations in Section 405.

416.2.2.2 Required strength of connections and adjacent regions shall be calculated in accordance with the analysis procedures in Section 406.

416.2.3 Design Strength

416.2.3.1 For each applicable load combination, design strengths of precast member connections shall satisfy

$$\phi S_n \geq U \quad (416.2.3.1)$$

416.2.3.2 ϕ shall be determined in accordance with Section 421.2.

416.2.3.3 At the contact surface between supported and supporting members, or between a supported or supporting member and an intermediate bearing element, nominal bearing strength for concrete surfaces, B_n , shall be calculated in accordance with Section 422.8. B_n shall be the lesser of the nominal concrete bearing strengths for the supported or supporting member surface, and shall not exceed the strength of intermediate bearing elements, if present.

416.2.3.4 If shear is the primary result of imposed loading and shear transfer occurs across a given plane, it shall be permitted to calculate V_n in accordance with the shear friction provisions in Section 422.9.

416.2.4 Minimum Connection Strength and Integrity Tie Requirements

416.2.4.1 Unless where the provisions of Section 416.2.5 govern, longitudinal and transverse integrity ties shall connect precast members to a lateral force-resisting system, and vertical integrity ties shall be provided in accordance with Section 416.2.4.3 to connect adjacent floor and roof levels.

416.2.4.2 Where precast members form floor or roof diaphragms, the connections between the diaphragm and those members being laterally supported by the diaphragm shall have a nominal tensile strength of not less than 4.4 kN per linear m.

416.2.4.3 Vertical integrity ties shall be provided at horizontal joints between all vertical precast structural members, except cladding, and shall satisfy (a) or (b):

- a. Concrete connections between precast columns shall have vertical integrity ties, with a nominal tensile strength of at least $1.4A_g$, in N, where A_g is the gross area of the column, mm^2 . For columns with a larger cross-section than required by consideration of loading, a reduced effective area based on the cross section required shall be permitted. The reduced effective area shall be at least one-half the gross area of the column;
- b. The Connections between precast wall panels shall have at least two vertical integrity ties, with a nominal tensile strength of at least 44 kN per tie.

416.2.5 Integrity Tie Requirements for Precast Concrete Bearing Wall Structures Three Storeys or More in Height

416.2.5.1 Integrity ties in floor and roof systems shall satisfy (a) through (f):

- a. Longitudinal and transverse integrity ties shall be provided in floor and roof systems to provide a nominal tensile strength of at least 22 kN per meter of width or length;
- b. Longitudinal and transverse integrity ties shall be provided over interior wall supports and between the floor or roof system and exterior walls;
- c. Longitudinal and transverse integrity ties shall be positioned in or within 600 mm. of the plane of the floor or roof system;
- d. Longitudinal integrity ties shall be oriented parallel to floor or roof slab spans and shall be spaced not greater than 3.0 m on center. Provisions shall be made to transfer forces around openings;
- e. Transverse integrity ties shall be oriented perpendicular to floor or roof slab spans and shall be spaced not greater than the bearing wall spacing;
- f. Integrity ties at the perimeter of each floor and roof, within 1.2 m of the edge, shall provide a nominal tensile strength of at least 71 kN.

416.2.5.2 Vertical integrity ties shall satisfy (a) through (c):

- a. Integrity ties shall be provided in all wall panels and shall be continuous over the height of the building;
- b. Integrity ties shall provide a nominal tensile strength of at least 44 kN per horizontal meter of wall;
- c. At least two integrity ties shall be provided in each wall panel.

416.2.6 Minimum Dimensions at Bearing Connections

416.2.6.1 Dimensions of bearing connections shall satisfy Section 416.2.6.2 or 416.2.6.3 unless shown by analysis or test that lesser dimensions will not impair performance.

416.2.6.2 For precast slabs, beams, or stemmed members, minimum design dimensions from the face of support to end of precast member in the direction of the span, considering specified tolerances, shall be in accordance with Table 416.2.6.2.

Table 416.2.6.2
Minimum Design Dimensions from Face of Support to End of Precast Member

Member type	Minimum distance, mm.	
Solid or hollow-core slab	Greater of:	$\ell_n/180$
		50
Beam or stemmed member	Greater of:	$\ell_n/180$
		75

416.2.6.3 Bearing pads adjacent to unarmored faces shall be set back from the face of the support and the end of the supported member a distance not less than 12 mm. or the chamfer dimension at a chamfered face.

416.3 Connections to Foundations

416.3.1 General

416.3.1.1 Factored forces and moments at base of columns, walls, or pedestals shall be transferred to supporting foundations by bearing on concrete and by reinforcement, dowels, anchor bolts, or mechanical connectors.

416.3.1.2 Reinforcement, dowels, or mechanical connectors between a supported member and foundation shall be designed to transfer (a) and (b):

- a. Compressive forces that exceed the lesser of the concrete bearing strengths of either the supported member or the foundation, calculated in accordance with Section 422.8;
- b. Any calculated tensile force across the interface.

416.3.1.3 At the base of a composite column with a structural steel core, (a) or (b) shall be satisfied:

- a. Base of structural steel section shall be designed to transfer the total factored forces from the entire composite member to the foundation;
- b. Base of structural steel section shall be designed to transfer the factored forces from the steel core only, and the remainder of the total factored forces shall be transferred to the foundation by compression in the concrete and by reinforcement.

416.3.2 Required Strength

416.3.2.1 Factored forces and moments transferred to foundations shall be calculated in accordance with the factored load combinations in Section 405 and analysis procedures in Section 406.

416.3.3 Design Strength

416.3.3.1 Design strengths of connections between columns, walls, or pedestals and foundations shall satisfy Eq. 416.3.3.1 for each applicable load combination. For connections between precast members and foundations, requirements for vertical integrity ties in Section 416.2.4.3 or 416.2.5.2 shall be satisfied.

$$\phi S_n \geq U \quad (416.3.3.1)$$

where S_n is the nominal flexural, shear, axial, torsional, or bearing strength of the connection.

416.3.3.2 ϕ shall be determined in accordance with Section 421.2.

416.3.3.3 Combined moment and axial strength of connections shall be calculated in accordance with Section 422.4.

416.3.3.4 At the contact surface between a supported member and foundation, or between a supported member or foundation and an intermediate bearing element, nominal bearing strength, B_n , shall be calculated in accordance with Section 422.8 for concrete surfaces. B_n shall be the lesser of the nominal concrete bearing strengths for the supported member or foundation surface,

and shall not exceed the strength of intermediate bearing elements, if present.

416.3.3.5 At the contact surface between supported member and foundation, V_n shall be calculated in accordance with the shear friction provisions in Section 422.9 or by other appropriate means.

416.3.3.6 At the base of a precast column, pedestal, or wall, anchor bolts and anchors for mechanical connections shall be designed in accordance with Section 417. Forces developed during erection shall be considered.

416.3.3.7 At the base of a precast column, pedestal, or wall, mechanical connectors shall be designed to reach their design strength before anchorage failure or failure of surrounding concrete.

416.3.4 Minimum Reinforcement for Connections Between Cast-In-Place Members And Foundation

416.3.4.1 For connections between a cast-in-place column or pedestal and foundation, A_s crossing the interface shall be at least $0.005A_g$, where A_g is the gross area of the supported member.

416.3.4.2 For connections between a cast-in-place wall and foundation, area of vertical reinforcement crossing the interface shall satisfy Section 411.6.1.

416.3.5 Details for Connections between Cast-in-Place Members and Foundation

416.3.5.1 At the base of a cast-in-place column, pedestal, or wall, reinforcement required to satisfy Sections 416.3.3 and 416.3.4 shall be provided either by extending longitudinal bars into supporting foundation or by dowels.

416.3.5.2 Where moments are transferred to the foundation, reinforcement, dowels, or mechanical connectors shall satisfy Section 410.7.5 for splices.

416.3.5.3 If a pinned or rocker connection is used at the base of a cast-in-place column or pedestal, the connection to foundation shall satisfy Section 416.3.3.

416.3.5.4 At footings, it shall be permitted to lap splice 40 mm ϕ and 58 mm ϕ longitudinal bars, in compression only, with dowels to satisfy Section 416.3.3.1. Dowels shall satisfy (a) through (c):

- a. Dowels shall not be larger than 36 mm ϕ ;

- b. Dowels shall extend into supported member at least the greater of the development length of the longitudinal bars in compression, ℓ_{dc} , and the compression lap splice length of the dowels, ℓ_{sc} ;
- c. Dowels shall extend into the footing at least ℓ_{dc} of the dowels.

416.3.6 Details for Connections between Precast Members and Foundation

416.3.6.1 At the base of a precast column, pedestal or wall, the connection to the foundation shall satisfy Section 416.2.4.3 or 416.2.5.2.

416.3.6.2 If the applicable load combinations of Section 416.3.3 result in no tension at the base of precast walls, vertical integrity ties required by Section 416.2.4.3(b) shall be permitted to be developed into an adequately reinforced concrete slab-on-ground.

416.4 Horizontal Shear Transfer in Composite Concrete Flexural Members

416.4.1 General

416.4.1.1 In a composite concrete flexural member, horizontal shear forces shall be provided at contact surfaces of interconnected elements.

416.4.1.2 Where tension exists across any contact surface between interconnected concrete elements, horizontal shear transfer by contact shall be permitted only where transverse reinforcement is provided in accordance with Sections 416.4.6 and 416.4.7.

416.4.1.3 Surface preparation assumed for design shall be specified in the construction documents.

416.4.2 Required Strength

416.4.2.1 Factored forces transferred along the contact surface in composite concrete flexural members shall be calculated in accordance with the factored load combinations in Section 405.

416.4.2.2 Required strength shall be calculated in accordance with the analysis procedures in Section 406.

416.4.3 Design Strength

416.4.3.1 Design strength for horizontal shear transfer shall satisfy Eq. 416.4.3.1 at all locations along the horizontal interface in a composite concrete flexural member, unless Section 416.4.5 is satisfied:

$$\phi V_{nh} \geq V_u \quad (416.4.3.1)$$

where nominal horizontal shear strength, V_{nh} , is calculated in accordance with Section 416.4.4.

416.4.3.2 ϕ shall be determined in accordance with Section 421.2.

416.4.4 Nominal Horizontal Shear Strength

416.4.4.1 If $V_u > \phi(3.5b_v d)$, V_{nh} shall be taken as V_n calculated in accordance with Section 422.9.

where, b_v is the width of the contact surface, and d is in accordance with Section 416.4.4.3.

416.4.4.2 If $V_u \leq \phi(3.5b_v d)$, V_{nh} shall be taken as V_n calculated in accordance with Table 416.4.4.2, where $A_{v,min}$ is in accordance with Section 416.4.6, b_v is the width of the contact surface, and d is in accordance with Section 416.4.4.3.

416.4.4.3 In Table 416.4.4.2, d shall be the distance from extreme compression fiber for the entire composite section to the centroid of prestressed and non-prestressed longitudinal tension reinforcement, if any, but need not be taken less than $0.80h$ for prestressed concrete members.

416.4.4.4 Transverse reinforcement in the previously cast concrete that extends into the cast-in-place concrete and is anchored on both sides of the interface shall be permitted to be included as ties for calculation of V_{nh} .

416.4.5 Alternative Method for Calculating Design Horizontal Shear Strength

416.4.5.1 As an alternative to Section 416.4.3.1, factored horizontal shear V_{uh} shall be calculated from the change in flexural compressive or tensile force in any segment of the composite concrete member, and Eq. 416.4.5.1 shall be satisfied at all locations along the horizontal interface:

$$\phi V_{nh} \geq V_{uh} \quad (416.4.5.1)$$

Nominal horizontal shear strength, V_{nh} , shall be calculated in accordance with Section 416.4.4.1 or 416.4.4.2, where area of contact surface shall be substituted for $b_v d$ and V_{uh} shall be substituted for V_u . Provisions shall be made to transfer the change in compressive or tensile force as horizontal shear force across the interface.

416.4.5.2 Where shear transfer reinforcement is designed to resist horizontal shear to satisfy Eq. 416.4.5.1, the tie area to tie spacing ratio along the member shall

approximately reflect the distribution of interface shear forces in the composite concrete flexural member.

416.4.5.3 Transverse reinforcement in a previously cast section that extends into the cast-in-place section and is anchored on both sides of the interface shall be permitted to be included as ties for calculation of V_{nh} .

Table 416.4.4.2
Nominal Horizontal Shear Strength

Shear transfer reinforcement	Contact surface preparation ^[1]	V_{nh} , N	
$A_v \geq A_{v,min}$	Concrete placed against hardened concrete intentionally roughened to a full amplitude of approximately 6 mm.	Lesser of:	$\lambda \left(1.8 + 0.6 \frac{A_v f_{yt}}{b_v s} \right) b_v d$ (a)
			$3.5 b_v d$ (b)
	Concrete placed against hardened concrete not intentionally roughened		$0.55 b_v d$ (c)
Other cases	Concrete placed against hardened concrete intentionally roughened		$0.55 b_v d$ (d)

[1] Concrete contact surface shall be clean and free of laitance

416.4.6 Minimum Reinforcement for Horizontal Shear Transfer

416.4.6.1 Where shear transfer reinforcement is designed to resist horizontal shear, $A_{v,min}$ shall be the greater of (a) and (b):

a. $0.062 \sqrt{f'_c} \frac{b_w s}{f_y}$

b. $0.35 \frac{b_w s}{f_y}$

416.4.7 Reinforcement Detailing for Horizontal Shear Transfer

416.4.7.1 Shear transfer reinforcement shall consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire reinforcement.

416.4.7.2 Where shear transfer reinforcement is designed to resist horizontal shear, longitudinal spacing of shear transfer reinforcement shall not exceed the lesser of 600 mm and four times the least dimension of the supported element.

416.4.7.3 Shear transfer reinforcement shall be developed in interconnected elements in accordance with Section 425.7.1.

416.5 Brackets and Corbels

416.5.1 General

416.5.1.1 Brackets and corbels with shear span-to-depth ratio $a_v/d \leq 1.0$ and with factored horizontal tensile force $N_{uc} \leq V_u$ shall be permitted to be designed in accordance with Section 416.5.

416.5.2 Dimensional Limits

416.5.2.1 Effective depth d for a bracket or corbel shall be calculated at the face of the support.

416.5.2.2 Overall depth of bracket or corbel at the outside edge of the bearing area shall be at least **0.5d**.

416.5.2.3 No part of the bearing area on a bracket or corbel shall project farther from the face of support than (a) or (b):

- a. End of the straight portion of the primary tension reinforcement;
- b. Interior face of the transverse anchor bar, if one is provided.

416.5.2.4 For normal-weight concrete, the bracket or corbel dimensions shall be selected such that V_u/ϕ shall not exceed the least of (a) through (c):

- a. $0.2f'_c b_w d$
- b. $(3.3 + 0.08f'_c) b_w d$
- c. $11b_w d$

416.5.2.5 For all-lightweight or sand-lightweight concrete, the bracket or corbel dimensions shall be selected such that V_u/ϕ shall not exceed the lesser of (a) and (b):

- a. $(0.2 - 0.07 \frac{a_v}{d}) f'_c b_w d$
- b. $(5.5 - 1.9 \frac{a_v}{d}) b_w d$

416.5.3 Required Strength

416.5.3.1 The section at the face of the support shall be designed to resist simultaneously the factored shear V_u , the factored horizontal tensile force N_{uc} , and the factored moment M_u given by $[V_u a_v + N_{uc}(h - d)]$.

416.5.3.2 Factored tensile force, N_{uc} , and shear, V_u , shall be the maximum values calculated in accordance with the factored load combinations in Section 405.

416.5.3.3 Required strength shall be calculated in accordance with the analysis procedures in Section 406, and the requirements in this section.

416.5.3.4 Horizontal tensile force acting on a bracket or corbel shall be treated as a live load when calculating N_{uc} , even if the tension results from restraint of creep, shrinkage, or temperature change.

416.5.3.5 Unless tensile forces are prevented from being applied to the bracket or corbel, N_{uc} shall be at least $0.2V_u$.

416.5.4 Design Strength

416.5.4.1 Design strength at all sections shall satisfy $\phi S_n \geq U$, including (a) through (c). Interaction between load effects shall be considered.

- a. $\phi N_n \geq N_{uc}$
- b. $\phi V_n \geq V_u$
- c. $\phi M_n \geq M_u$

416.5.4.2 ϕ shall be determined in accordance with Section 421.2.

416.5.4.3 Nominal tensile strength N_n provided by A_n shall be calculated by

$$N_n \geq A_n f_y \quad (416.5.4.3)$$

416.5.4.4 Nominal shear strength V_n provided by A_{vf} shall be calculated in accordance with provisions for shear-friction in Section 422.9, where A_{vf} is the area of reinforcement that crosses the assumed shear plane.

416.5.4.5 Nominal flexural strength M_n provided by A_f shall be calculated in accordance with the design assumptions in Section 422.2.

416.5.5 Reinforcement Limits

416.5.5.1 Area of primary tension reinforcement, A_{sc} , shall be at least the greatest of (a) through (c):

- a. $A_f + A_n$
- b. $(2/3)A_{vf} + A_n$
- c. $0.04(f'_c/f_y)(b_w d)$

416.5.5.2 Total area of closed stirrups or ties parallel to primary tension reinforcement, A_h , shall be at least:

$$A_h = 0.5(A_{sc} - A_n) \quad (416.5.5.2)$$

416.5.6 Reinforcement Detailing

416.5.6.1 Concrete cover shall be in accordance with Section 420.6.1.3.

416.5.6.2 Minimum spacing for deformed reinforcement shall be in accordance with Section 425.2.

416.5.6.3 At the front face of a bracket or corbel, primary tension reinforcement shall be anchored by (a), (b), or (c):

- a. A weld to a transverse bar of at least equal size that is designed to develop f_y of primary tension reinforcement;
- b. Bending the primary tension reinforcement back to form a horizontal loop;
- c. Other means of anchorage that develops f_y .

416.5.6.4 Primary tension reinforcement shall be developed at the face of the support.

416.5.6.5 Development of tension reinforcement shall account for distribution of stress in reinforcement that is not directly proportional to the bending moment.

416.5.6.6 Closed stirrups or ties shall be spaced such that A_h is uniformly distributed within $(2/3)d$ measured from the primary tension reinforcement.

SECTION 417 ANCHORING TO CONCRETE

417.1 Scope

417.1.1 This section provides design requirements for anchors in concrete used to transmit structural loads by means of tension, shear, or a combination of tension and shear between: (a) connected structural elements; or (b) safety-related attachments and structural elements. Safety levels specified are intended for in-service conditions, rather than for short-term handling and construction conditions.

417.1.2 This section applies to cast-in anchors and to post-installed expansion (torque-controlled and displacement-controlled), undercut, and adhesive anchors. Adhesive anchors shall be installed in concrete having a minimum age of 21 days at time of anchor installation. Specialty inserts, through-bolts, multiple anchors connected to a single steel plate at the embedded end of the anchors, grouted anchors, and direct anchors such as powder or pneumatic actuated nails or bolts are not included in the provisions of this section. Reinforcement used as part of the embedment shall be designed in accordance with other parts of this Chapter.

417.1.3 Design provisions are included for the following types of anchors:

- a. Headed studs and headed bolts having a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal to or exceeding $1.4N_p$, where N_p is given in Eq. 417.4.3.4;
- b. Hooked bolts having a geometry that has been demonstrated to result in a pullout strength without the benefit of friction in uncracked concrete equal to or exceeding $1.4N_p$, where N_p is given in Eq. 417.4.3.5;
- c. Post-installed expansion and undercut anchors that meet the assessment criteria of ACI 355.2;
- d. Adhesive anchors that meet the assessment criteria of ACI 355.4M.

417.1.4 Load Applications that are Predominantly High Cycle Fatigue or Impact Loads are not covered by this Section.

417.2 General

417.2.1 Anchors and anchor groups shall be designed for critical effects of factored loads as determined by elastic analysis. Plastic analysis approaches are permitted where nominal strength is controlled by ductile steel elements, provided that deformational compatibility is taken into account.

417.2.1.1 Anchor group effects shall be considered wherever two or more anchors have spacing less than the critical spacing as follows:

Failure Mode Under Investigation	Critical Spacing
Concrete breakout in tension	$3h_{ef}$
Bond strength in tension	$2c_{NA}$
Concrete breakout in shear	$2c_{a1}$

Only those anchors susceptible to the particular failure mode under investigation shall be included in the group.

417.2.2 The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in Section 405.3.

417.2.3 Seismic Design

417.2.3.1 Anchors in structures assigned to seismic zone 4 shall satisfy the additional requirements of Sections 417.2.3.2 through 417.2.3.7.

417.2.3.2 The provisions of this section do not apply to the design of anchors in plastic hinge zones of concrete structures under earthquake forces.

417.2.3.3 Post-installed anchors shall be qualified for earthquake loading in accordance with ACI 355.2 or ACI 355.4M. The pullout strength N_p and steel strength in shear V_{sa} of expansion and undercut anchors shall be based on the results of the ACI 355.2 Simulated Seismic Tests. For adhesive anchors, the steel strength in shear V_{sa} and the characteristic bond stresses τ_{uncr} and τ_{cr} shall be based on results of the ACI 355.4M Simulated Seismic Tests.

417.2.3.4 Requirements for Tensile Loading

417.2.3.4.1 Where the tensile component of the strength-level earthquake force applied to a single anchor or group of anchors is equal to or less than 20 percent of the total factored anchor tensile force associated with the same load combination, it shall be permitted to design a single anchor or group of anchors to satisfy Section 417.4 and the tensile strength requirements of Section 417.3.1.1.

417.2.3.4.2 Where the tensile component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor tensile force associated with the same load combination, anchors and their attachments shall be designed in accordance with Section 417.2.3.4.3. The anchor design tensile strength shall be determined in accordance with Section 417.2.3.4.4.

417.2.3.4.3 Anchors and their attachments shall satisfy one of options (a) through (d):

- a. For single anchors, the concrete-governed strength shall be greater than the steel strength of the anchor. For anchor groups, the ratio of the tensile load on the most highly stressed anchor to the steel strength of that anchor shall be equal to or greater than the ratio of the tensile load on tension loaded anchors to the concrete-governed strength of those anchors. In each case:
 - i. The steel strength shall be taken as 1.2 times the nominal steel strength of the anchor;
 - ii. The concrete-governed strength shall be taken as the nominal strength considering pullout, side face blowout, concrete breakout, and bond strength as applicable. For consideration of pullout in groups, the ratio shall be calculated for the most highly stressed anchor.

In addition, the following shall be satisfied:

- iii. Anchors shall transmit tensile loads via a ductile steel element with a stretch length of at least eight anchor diameters unless otherwise determined by analysis;
- iv. Where anchors are subject to load reversals, the anchor shall be protected against buckling;
- v. Where connections are threaded and the ductile steel elements are not threaded over their entire length, the ratio of f_{uta}/f_{ya} shall not be less than 1.3 unless the threaded portions are upset.

The upset portions shall not be included in the stretch length;

- vi. Deformed reinforcing bars used as ductile steel elements to resist earthquake effects shall be limited to ASTM A615M Grades 280 and 420 satisfying the requirements of Section 420.2.2.5(b) or ASTM A706M Grade 420.
- b. The anchor or group of anchors shall be designed for the maximum tension that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in tension, flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects for the attachment. The anchor design tensile strength shall be calculated from Section 417.2.3.4.4;
- c. The anchor or group of anchors shall be designed for the maximum tension that can be transmitted to the anchors by a non-yielding attachment. The anchor design tensile strength shall be calculated from Section 417.2.3.4.4;
- d. The anchor or group of anchors shall be designed for the maximum tension obtained from design load combinations that include E , with E increased by Ω_o . The anchor design tensile strength shall satisfy the tensile strength requirements of Section 417.2.3.4.4.

417.2.3.4.4 The anchor design tensile strength for resisting earthquake forces shall be determined from consideration of (a) through (e) for the failure modes given in Table 417.3.1.1 assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked:

- a. ϕN_{sa} for a single anchor, or for the most highly stressed individual anchor in a group of anchors;
- b. $0.75\phi N_{cb}$ or $0.75\phi N_{cbg}$, except that N_{cb} or N_{cbg} need not be calculated where anchor reinforcement satisfying Section 417.4.2.9 is provided;
- c. $0.75\phi N_{pn}$ for a single anchor, or for the most highly stressed individual anchor in a group of anchors;
- d. $0.75\phi N_{sb}$ or $0.75\phi N_{sbg}$;
- e. $0.75\phi N_a$ or $0.75\phi N_{ag}$.

where ϕ is in accordance with Section 417.3.3.

417.2.3.4.5 Where anchor reinforcement is provided in accordance with Section 417.4.2.9, no reduction in design tensile strength beyond that specified in Section 417.4.2.9 shall be required.

417.2.3.5 Requirements for Shear Loading

417.2.3.5.1 Where the shear component of the strength-level earthquake force applied to the anchor or group of anchors is equal to or less than 20 percent of the total factored anchor shear force associated with the same load combination, it shall be permitted to design the anchor or group of anchors to satisfy Section 417.5 and the shear strength requirements of Section 417.3.1.1.

417.2.3.5.2 Where the shear component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor shear force associated with the same load combination, anchors and their attachments shall be designed in accordance with Section 417.2.3.5.3. The anchor design shear strength for resisting earthquake forces shall be determined in accordance with Section 417.5.

417.2.3.5.3 Anchors and their attachments shall be designed using one of options (a) through (c):

- a. The anchor or group of anchors shall be designed for the maximum shear that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects in the attachment;
- b. The anchor or group of anchors shall be designed for the maximum shear that can be transmitted to the anchors by a non-yielding attachment;
- c. The anchor or group of anchors shall be designed for the maximum shear obtained from design load combinations that include E , with E increased by Ω_o . The anchor design shear strength shall satisfy the shear strength requirements of Section 417.3.1.1.

417.2.3.5.4 Where anchor reinforcement is provided in accordance with Section 417.5.2.9, no reduction in design shear strength beyond that specified in Section 417.5.2.9 shall be required.

417.2.3.6 Single anchors or groups of anchors that are subjected to both tension and shear forces shall be designed to satisfy the requirements of Section 417.6, with the anchor design tensile strength calculated from Section 417.2.3.4.4.

417.2.3.7 Anchor reinforcement used in structures assigned to seismic zone 4 shall be deformed reinforcement and shall be limited to ASTM A615M Grades 280 and 420 satisfying the requirements of Section 420.2.2.5(b) or ASTM A706M Grade 420.

417.2.4 Adhesive anchors installed horizontally or upwardly inclined shall be qualified in accordance with ACI 355.4M requirements for sensitivity to installation direction.

417.2.5 For adhesive anchors subjected to sustained tension loading, Section 417.3.1.2 shall be satisfied. For groups of adhesive anchors, Eq. 417.3.1.2 shall be satisfied for the anchor that resists the highest sustained tension load. Installer certification and inspection requirements for horizontal and upwardly inclined adhesive anchors subjected to sustained tension loading shall be in accordance with Sections 417.8.2.2 through 417.8.2.4.

417.2.6 Modification factor λ_a for lightweight concrete shall be taken as:

Cast-in and undercut anchor concrete failure **1.0 λ**

Expansion and adhesive anchor concrete failure **0.8 λ**

Adhesive anchor bond failure per Eq. 417.4.5.2 **0.6 λ**

where λ is determined in accordance with Section 419.2.4. It shall be permitted to use an alternative value of λ_a where tests have been performed and evaluated in accordance with ACI 355.2 or ACI 355.4M.

417.2.7 The values of f'_c used for calculation purposes in this chapter shall not exceed 70 MPa for cast-in anchors, and 55 MPa for post-installed anchors. Testing is required for post-installed anchors when used in concrete with f'_c greater than 55 MPa.

417.3 General Requirements for Strength of Anchors

417.3.1 Strength design of anchors shall be based either on computation using design models that satisfy the requirements of Section 417.3.2, or on test evaluation using the 5 percent fractile of applicable test results for the following:

- Steel strength of anchor in tension (Section 417.4.1);
- Concrete breakout strength of anchor in tension (Section 417.4.2);

- Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (Section 417.4.3);
- Concrete side-face blowout strength of headed anchor in tension (Section 417.4.4);
- Bond strength of adhesive anchor in tension (Section 417.4.5);
- Steel strength of anchor in shear (Section 417.5.1);
- Concrete breakout strength of anchor in shear (Section 417.5.2);
- Concrete pryout strength of anchor in shear (Section 417.5.3).

In addition, anchors shall satisfy the required edge distances, spacings, and thicknesses to preclude splitting failure, as required in Section 417.7.

417.3.1.1 The design of anchors shall be in accordance with Table 417.3.1.1. In addition, the design of anchors shall satisfy Section 417.2.3 for earthquake loading and Section 417.3.1.2 for adhesive anchors subject to sustained tensile loading.

417.3.1.2 For the design of adhesive anchors to resist sustained tensions loads, in addition to Section 417.3.1.1, Eq. 417.3.1.2 shall be satisfied.

$$0.55\phi N_{ba} \geq N_{ua,s} \quad (417.3.1.2)$$

where N_{ba} is determined in accordance with Section 417.4.5.2.

417.3.1.3 When both N_{ua} and V_{ua} are present, interaction effects shall be considered using an interaction expression that results in computation of strength in substantial agreement with results of comprehensive tests. This requirement shall be considered satisfied by Section 417.6.

417.3.2 The nominal strength for any anchor or group of anchors shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests. The materials used in the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5 percent fractile of the basic individual anchor strength. For nominal strengths related to concrete strength, modifications for size effects, the number of anchors, the effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loadings of anchor groups, and presence or absence of cracking shall

be taken into account. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

Table 417.3.1.1 Required Strength of Anchors, Except as Noted in Section 417.2.3

Failure mode	Single anchor	Anchor group ^[1]	
		Individual anchor in a group	Anchors as a group
Steel strength in tension (Section 417.4.1)	$\phi N_{sa} \geq N_{ua}$	$\phi N_{sa} \geq N_{ua,i}$	
Concrete breakout strength in tension (Section 417.4.2)	$\phi N_{cb} \geq N_{ua}$		$\phi N_{cbg} \geq N_{ua,g}$
Pullout strength in tension (Section 417.4.3)	$\phi N_{pn} \geq N_{ua}$	$\phi N_{pn} \geq N_{ua,i}$	
Concrete side-face blowout strength in tension (Section 417.4.4)	$\phi N_{sb} \geq N_{ua}$		$\phi N_{sbg} \geq N_{ua,g}$
Bond strength of adhesive anchor intension (Section 417.4.5)	$\phi N_a \geq N_{ua}$		$\phi N_{ag} \geq N_{ua,g}$
Steel strength in shear (Section 417.5.1)	$\phi V_{sa} \geq V_{ua}$	$\phi V_{sa} \geq V_{ua,i}$	
Concrete breakout strength in shear (Section 417.5.2)	$\phi V_{cb} \geq V_{ua}$		$\phi V_{cbg} \geq V_{ua,g}$
Concrete pryout strength in shear (Section 417.5.3)	$\phi V_{cp} \geq V_{ua}$		$\phi V_{cpg} \geq V_{ua,g}$

^[1] Required strengths for steel and pullout failure modes shall be calculated for the most highly stressed anchor in the group.

417.3.2.1 The effect of reinforcement provided to restrain the concrete breakout shall be permitted to be included in the design models used to satisfy Section 417.3.2. Where anchor reinforcement is provided in accordance with Sections 417.4.2.9 and 417.5.2.9, calculation of the concrete breakout strength in accordance with Sections 417.4.2 and 417.5.2 is not required.

417.3.2.2 For anchors with diameters not exceeding 100mm, the concrete breakout strength requirements shall be considered satisfied by the design procedure of Sections 417.4.2 and 417.5.2.

417.3.2.3 For adhesive anchors with embedment depths $4d_a \leq h_{ef} \leq 20d_a$, the bond strength requirements shall be considered satisfied by the design procedure of Section 417.4.5.

417.3.3 Strength reduction factor ϕ for anchors in concrete shall be as follows when the load combinations of Section 405.3 are used:

- a. Anchor governed by strength of a ductile steel element
 - i. Tension loads0.75
 - ii. Shear loads0.65
- b. Anchor governed by strength of a brittle steel element
 - i. Tension loads0.65

ii. Shear loads0.60

- c. Anchor governed by concrete breakout, side-face blowout, pullout, or pryout strength

Condition A Condition B

i. Shear loads0.750.70

ii. Tension loads
Cast-in headed studs,
headed bolts,
or
hooked bolts0.750.70

Post-installed anchors with category as determined from
ACI 355.2 or ACI 355.4M

Condition A Condition B

Category 10.750.65
(Low sensitivity to installation and high reliability)

Category 20.650.55
(Medium sensitivity to installation and medium reliability)

Category 30.550.45
(High sensitivity to installation and lower reliability)

Condition A applies where supplementary reinforcement is present except for pullout and pryout strengths.

Condition B applies where supplementary reinforcement is not present, and for pullout or pryout strength.

417.4 Design Requirements for Tensile Loading

417.4.1 Steel Strength of Anchor in Tension

417.4.1.1 The nominal strength of an anchor in tension as governed by the steel, N_{sa} , shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

417.4.1.2 The nominal strength of an anchor in tension, N_{sa} , shall not exceed

$$N_{sa} = A_{se,N} f_{uta} \quad (417.4.1.2)$$

where $A_{se,N}$ is the effective cross-sectional area of an anchor in tension, mm^2 , and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 860 MPa.

417.4.2 Concrete Breakout Strength of Anchor in Tension

417.4.2.1 The nominal concrete breakout strength in tension, N_{cb} of a single anchor or N_{cbg} of a group of anchors, shall not exceed:

a. For a single anchor;

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad (417.4.2.1a)$$

b. For a group of anchors;

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad (417.4.2.1b)$$

Factors $\psi_{ec,N}$, $\psi_{ed,N}$, $\psi_{c,N}$, and $\psi_{cp,N}$ are defined in Sections 417.4.2.4, 417.4.2.5, 417.4.2.6, and 417.4.2.7, respectively. A_{Nc} is the projected concrete failure area of a single anchor or group of anchors that shall be approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward $1.5h_{ef}$ from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors. A_{Nc} shall not exceed nA_{Nco} , where n is the number of anchors in the group that resist tension. nA_{Nco} is the projected concrete failure area of a single anchor with an edge distance equal to or greater than $1.5h_{ef}$

$$A_{Nco} = 9h_{ef}^2 \quad (417.4.2.1c)$$

417.4.2.2 The basic concrete breakout strength of a single anchor in tension in cracked concrete, N_b , shall not exceed

$$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} \quad (417.4.2.2a)$$

where $k_c = 10$ for cast-in anchors, $k_c = 17$ for post-installed anchors.

The value of k_c for post-installed anchors shall be permitted to be increased above 17 based on ACI 355.2 or ACI 355.4M product-specific tests, but shall not exceed 10.

Alternatively, for cast-in headed studs and headed bolts with $280 \text{ mm} \leq h_{ef} \leq 635 \text{ mm}$, N_b shall not exceed

$$N_b = 3.9 \lambda_a \sqrt{f'_c} h_{ef}^{5/3} \quad (417.4.2.2b)$$

417.4.2.3 Where anchors are located less than $1.5h_{ef}$ from three or more edges, the value of h_{ef} used for the calculation of A_{Nc} in accordance with Section 417.4.2.1, as well as for the equations in Sections 417.4.2.1 through 417.4.2.5 shall be the larger of $(c_{a,max})/1.5$ and $s/3$, where s is the maximum spacing between anchors within the group.

417.4.2.4 The modification factor for anchor groups loaded eccentrically in tension, $\psi_{ec,N}$, shall be calculated as

$$\psi_{ec,N} = \frac{1}{\left(1 + \frac{2e'_N}{3h_{ef}}\right)} \quad (417.4.2.4)$$

but $\psi_{ec,N}$ shall not be taken greater than 1.0. If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension shall be considered when determining the eccentricity e'_N for use in Eq. 417.4.2.4 and for the calculation of N_{cbg} according to Eq. 417.4.2.1b.

In the case where eccentric loading exists about two axes, the modification factor, $\psi_{ec,N}$ shall be calculated for each axis individually and the product of these factors used as $\psi_{ec,N}$ in Eq. 417.4.2.1b.

417.4.2.5 The modification factor for edge effects for single anchors or anchor groups loaded in tension, $\psi_{ed,N}$, shall be calculated as:

If $c_{a,min} \geq 1.5h_{ef}$, then when $\psi_{ed,N} = 1.0$ (417.4.2.5a)

If $c_{a,min} < 1.5h_{ef}$, then $\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5h_{ef}}$ (417.4.2.5b)

417.4.2.6 For anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, the following modification factor shall be permitted:

- $\psi_{c,N} = 1.25$ for cast-in anchors;
- $\psi_{c,N} = 1.4$ for post-installed anchors, where the value of k_c used in Eq. 417.4.2.2a is 17;

Where the value of k_c used in Eq. 417.4.2.2a is taken from the ACI 355.2 or ACI 355.4M product evaluation report for post-installed anchors qualified for use in both cracked and uncracked concrete, the values of k_c and $\psi_{c,N}$ shall be based on the ACI 355.2 or ACI 355.4M product evaluation report.

Where the value of k_c used in Eq. 417.4.2.2a is taken from the ACI 355.2 or ACI 355.4M product evaluation report for post-installed anchors qualified for use in uncracked concrete, $\psi_{c,N}$ shall be taken as 1.0.

When analysis indicates cracking at service load levels, $\psi_{c,N}$ shall be taken as 1.0 for both cast-in anchors and post-installed anchors. Post-installed anchors shall be qualified for use in cracked concrete in accordance with ACI 355.2 or ACI 355.4M. The cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with Section 424.3.2, or equivalent crack control shall be provided by confining reinforcement.

417.4.2.7 The modification factor for post-installed anchors designed for uncracked concrete in accordance with Section 417.4.2.6 without supplementary reinforcement to control splitting, $\psi_{cp,N}$, shall be calculated as follows using the critical distance c_{ac} as defined in Section 417.7.6.

If $c_{a,min} \geq c_{ac}$, then when $\psi_{cp,N} = 1.0$ (417.4.2.7a)

If $c_{a,min} < c_{ac}$, then when $\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}}$ (417.4.2.5b)

but $\psi_{cp,N}$ determined from Eq. 417.4.2.7b shall not be taken less than $1.5h_{ef}/c_{ac}$, where the critical distance c_{ac} is defined in Section 417.7.6.

For all other cases, including cast-in anchors, $\psi_{cp,N}$ shall be taken as 1.0.

417.4.2.8 Where an additional plate or washer is added at the head of the anchor, it shall be permitted to calculate the projected area of the failure surface by projecting the failure surface outward $1.5h_{ef}$ from the effective perimeter of the plate or washer. The effective perimeter shall not exceed the value at a section projected outward more than the thickness of the washer or plate from the outer edge of the head of the anchor.

417.4.2.9 Where anchor reinforcement is developed in accordance with Section 425 on both sides of the breakout surface, the design strength of the anchor reinforcement shall be permitted to be used instead of the concrete breakout strength in determining ϕN_n . A strength reduction factor of 0.75 shall be used in the design of the anchor reinforcement.

417.4.3 Pullout Strength of Cast-In, Post-Installed Expansion and Undercut Anchors in Tension

417.4.3.1 The nominal pullout strength of a single cast-in, post-installed expansion, and post-installed undercut anchor in tension, N_{pn} , shall not exceed

$$N_{pn} = \psi_{c,p} N_p \quad (417.4.3.1)$$

where $\psi_{c,p}$ is defined in Section 417.4.3.6.

417.4.3.2 For post-installed expansion and undercut anchors, the values of N_p shall be based on the 5 percent fractile of results of tests performed and evaluated according to ACI 355.2. It is not permissible to calculate the pullout strength in tension for such anchors.

417.4.3.3 For single cast-in headed studs and headed bolts, it shall be permitted to evaluate the pullout strength in tension using Section 417.4.3.4. For single J- or L-bolts, it shall be permitted to evaluate the pullout strength in tension using Section 417.4.3.5. Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

417.4.3.4 The pullout strength in tension of a single headed stud or headed bolt, N_p , for use in Eq. 417.4.3.1, shall not exceed:

$$N_p = 8A_{brg}f'_c \quad (417.4.3.4)$$

417.4.3.5 The pullout strength in tension of a single hooked bolt, N_p , for use in Eq. 417.4.3.1 shall not exceed:

$$N_p = 0.9f_c e_h d_a \quad (417.4.3.5)$$

where $d_a \leq e_h \leq 4.5d_a$.

417.4.3.6 For an anchor located in a region of a concrete member where analysis indicates no cracking at service load levels, the following modification factor shall be permitted:

$$\psi_{c,p} = 1.4 \quad (417.4.3.6)$$

Where analysis indicates cracking at service load levels, $\psi_{c,p}$ shall be taken as 1.0.

417.4.4 Concrete Side-Face Blowout Strength of a Headed Anchor in Tension

417.4.4.1 For a single headed anchor with deep embedment close to an edge ($h_{ef} > 2.5c_{a1}$), the nominal side-face blowout strength, N_{sb} , shall not exceed:

$$N_{sb} = (13c_{a1} \sqrt{A_{brg}}) \lambda_a \sqrt{f'_c} \quad (417.4.4.1)$$

If c_{a2} for the single headed anchor is less than $3c_{a1}$, the value of N_{sb} shall be multiplied by the factor $(1 + c_{a2}/c_{a1})/4$ where $1.0 \leq c_{a2}/c_{a1} \leq 3.0$.

417.4.4.2 For multiple headed anchors with deep embedment close to an edge ($h_{ef} > 2.5c_{a1}$) and anchor spacing less than $6c_{a1}$, the nominal strength of those anchors susceptible to a side-face blowout failure N_{sbg} shall not exceed:

$$N_{sbg} = \left(1 + \frac{s}{6c_{a1}}\right) N_{sb} \quad (417.4.4.2)$$

where s is the distance between the outer anchors along the edge, and N_{sb} is obtained from Eq. 417.4.4.1 without modification for a perpendicular edge distance.

417.4.5 Bond Strength of Adhesive Anchor in Tension

417.4.5.1 The nominal bond strength in tension, N_a of a single adhesive anchor or N_{ag} of a group of adhesive anchors, shall not exceed

a. For a single adhesive anchor:

$$N_a = \frac{A_{Na}}{A_{Na0}} \psi_{ec,Na} \psi_{ed,Na} \psi_{cp,Na} N_{ba} \quad (417.4.5.1a)$$

b. For a group of adhesive anchors:

$$N_{ag} = \frac{A_{Na}}{A_{Na0}} \psi_{ec,Na} \psi_{ed,Na} \psi_{cp,Na} N_{ba} \quad (417.4.5.1b)$$

Factors $\psi_{ec,Na}$, $\psi_{ed,Na}$, and $\psi_{cp,Na}$ are defined in Sections 417.4.5.3, 417.4.5.4, and 417.4.5.5, respectively. A_{Na} is the projected influence area of a single adhesive anchor or group of adhesive anchors that shall be approximated as a rectilinear area that projects outward a distance c_{Na} from the centerline of the adhesive anchor, or in the case of a group of adhesive anchors, from a line through a row of adjacent adhesive anchors. A_{Na} shall not exceed nA_{Na0} , where n is the number of adhesive anchors in the group that resist tension loads. A_{Na0} is the projected influence area of a single adhesive anchor with an edge distance equal to or greater than c_{Na} :

$$A_{Na0} = (2c_{Na})^2 \quad (417.4.5.1c)$$

where

$$c_{Na} = 10d_a \sqrt{\frac{\tau_{uncr}}{7.6}} \quad (417.4.5.1d)$$

and constant 7.6 carries the unit of MPa.

417.4.5.2 The basic bond strength of a single adhesive anchor in tension in cracked concrete, N_{ba} , shall not exceed

$$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef} \quad (417.4.5.2)$$

The characteristic bond stress, τ_{cr} , shall be taken as the 5 percent fractile of results of tests performed and evaluated according to ACI 355.4M.

Where analysis indicates cracking at service load levels, adhesive anchors shall be qualified for use in cracked concrete in accordance with ACI 355.4M.

For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} shall be permitted to be used in place of τ_{cr} in Eq. 417.4.5.2 and shall be taken as the 5 percent fractile of results of tests performed and evaluated according to ACI 355.4M.

It shall be permitted to use the minimum characteristic bond stress values in Table 417.4.5.2 provided (a) through (e) are satisfied:

a. Anchors shall meet the requirements of ACI 355.4M;

- b. Anchors shall be installed in holes drilled with a rotary impact drill or rock drill;
- c. Concrete at time of anchor installation shall have a minimum compressive strength of 17 MPa;
- d. Concrete at time of anchor installation shall have a minimum age of 21 days;
- e. Concrete temperature at time of anchor installation shall be at least 10°C.

Table 417.4.5.2 Minimum Characteristic Bond Stresses^{[1][2]}

Installation and service environment	Moisture content of concrete at time of anchor installation	Peak in-service Temperature of concrete, °C	τ_{cr} , MPa	τ_{uner} , MPa
Outdoor	Dry to fully saturated	79	1.4	4.5
Indoor	Dry	43	2.1	7.0

^[1] Where anchor design includes sustained tension loading, multiply values of τ_{cr} and τ_{uner} by 0.4.

^[2] Where anchor design includes earthquake loads for structures assigned to seismic zone 4, multiply values of τ_{cr} by 0.8 and τ_{uner} by 0.4.

417.4.5.3 The modification factor for adhesive anchor groups loaded eccentrically in tension, $\psi_{ec,Na}$, shall be calculated as:

$$\psi_{ec,Na} = \frac{1}{\left(1 + \frac{e'_N}{c_{Na}}\right)} \quad (417.4.5.3)$$

but $\psi_{ec,Na}$ shall not be taken greater than 1.0.

If the loading on an adhesive anchor group is such that only some adhesive anchors are in tension, only those adhesive anchors that are in tension shall be considered when determining the eccentricity e'_N for use in Eq. 417.4.5.3 and for the calculation of N_{ag} according to Eq. 417.4.5.1b.

In the case where eccentric loading exists about two orthogonal axes, the modification factor, $\psi_{ec,Na}$, shall be calculated for each axis individually and the product of these factors used as $\psi_{ec,Na}$ in Eq. 417.4.5.1b.

417.4.5.4 The modification factor for edge effects for single adhesive anchors or adhesive anchor groups loaded in tension, $\psi_{ed,Na}$, shall be calculated as:

If $c_{a,min} \geq c_{Na}$,

$$\text{then } \psi_{ed,Na} = 1.0 \quad (417.4.5.4a)$$

If $c_{a,min} < c_{Na}$,

$$\text{then } \psi_{ed,Na} = 0.7 + 0.3 \frac{c_{a,min}}{c_{Na}} \quad (417.4.5.4b)$$

417.4.5.5 The modification factor for adhesive anchors designed for uncracked concrete in accordance with Section 417.4.5.2 without supplementary reinforcement to control splitting, $\psi_{cp,Na}$, shall be calculated as:

If $c_{a,min} \geq c_{ac}$,

$$\text{then } \psi_{cp,Na} = 1.0 \quad (417.4.5.5a)$$

If $c_{a,min} < c_{ac}$,

$$\text{then } \psi_{ed,Na} = 0.7 + 0.3 \frac{c_{a,min}}{c_{Na}} \quad (417.4.5.4b)$$

but $\psi_{cp,Na}$ determined from Eq. 417.4.5.5b shall not be taken less than c_{Na}/c_{ac} , where the critical edge distance, c_{ac} , is defined in Section 417.7.6. For all other cases, $\psi_{cp,Na}$ shall be taken as 1.0.

417.5 Design Requirements for Shear Loading

417.5.1 Steel Strength of Anchor in Shear

417.5.1.1 The nominal strength of an anchor in shear as governed by steel, V_{sa} , shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor. Where concrete breakout is a potential failure mode, the required steel shear strength shall be consistent with the assumed breakout surface.

417.5.1.2 The nominal strength of an anchor in shear, V_{sa} , shall not exceed (a) through (c):

- a. For cast-in headed stud anchor;

$$V_{sa} = A_{se,V} f_{uta} \quad (417.5.1.2a)$$

where $A_{se,V}$ is the effective cross-sectional area of an anchor in shear, mm², and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 860 MPa.

- b. For cast-in headed bolt and hooked bolt anchors and for post-installed anchors where sleeves do not extend through the shear plane;

$$V_{sa} = 0.6A_{se,V}f_{uta} \quad (417.5.1.2b)$$

where $A_{se,V}$ is the effective cross-sectional area of an anchor in shear, mm^2 , and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 860 MPa.

- c. For post-installed anchors where sleeves extend through the shear plane, V_{sa} shall be based on the results of tests performed and evaluated according to ACI 355.2. Alternatively, Eq. 417.5.1.2b shall be permitted to be used.

417.5.1.3 Where anchors are used with built-up grout pads, the nominal strengths of Section 417.5.1.2 shall be multiplied by a factor 0.80.

417.5.2 Concrete Breakout Strength of Anchor in Shear

417.5.2.1 The nominal concrete breakout strength in shear, V_{cb} of a single anchor or V_{cbg} of a group of anchors, shall not exceed:

- a. For shear force perpendicular to the edge on a single anchor;

$$V_{cb} = \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b \quad (417.5.2.1a)$$

- b. For shear force perpendicular to the edge on a group of anchors;

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b \quad (417.5.2.1b)$$

- c. For shear force parallel to an edge, V_{cb} or V_{cbg} shall be permitted to be twice the value of the shear force determined from Eq. 417.5.2.1a or 417.5.2.1b, respectively, with the shear force assumed to act perpendicular to the edge and with $\psi_{ed,V}$ taken equal to 1.0;
- d. For anchors located at a corner, the limiting nominal concrete breakout strength shall be determined for each edge, and the minimum value shall be used.

Factors $\psi_{ec,V}$, $\psi_{ed,V}$, $\psi_{c,V}$, and $\psi_{h,V}$ are defined in Sections 417.5.2.5, 417.5.2.6, 417.5.2.7, and 417.5.2.8, respectively. V_b is the basic concrete breakout strength value for a single anchor. A_{Vc} is the projected area of the failure surface on the side of the concrete member at its

edge for a single anchor or a group of anchors. It shall be permitted to evaluate A_{Vc} as the base of a truncated half-pyramid projected on the side face of the member where the top of the half-pyramid is given by the axis of the anchor row selected as critical. The value of c_{a1} shall be taken as the distance from the edge to this axis. A_{Vc} shall not exceed nA_{Vco} , where n is the number of anchors in the group.

A_{Vco} is the projected area for a single anchor in a deep member with a distance from edges equal or greater than $1.5c_{a1}$ in the direction perpendicular to the shear force. It shall be permitted to evaluate A_{Vco} as the base of a half pyramid with a side length parallel to the edge of $3c_{a1}$ and a depth of $1.5c_{a1}$

$$A_{Vco} = 4.5(c_{a1})^2 \quad (417.5.2.1c)$$

Where anchors are located at varying distances from the edge and the anchors are welded to the attachment so as to distribute the force to all anchors, it shall be permitted to evaluate the strength based on the distance to the farthest row of anchors from the edge. In this case, it shall be permitted to base the value of c_{a1} on the distance from the edge to the axis of the farthest anchor row that is selected as critical, and all of the shear shall be assumed to be carried by this critical anchor row alone.

417.5.2.2 The basic concrete breakout strength in shear of a single anchor in cracked concrete, V_b , shall be the smaller of (a) and (b):

$$a. \quad V_b = \left(0.6 \left(\frac{\ell_e}{d_a} \right)^{0.2} \sqrt{d_a} \right) \lambda_a \sqrt{f'_c} (c_{a1})^{1.5} \quad (417.5.2.2a)$$

where ℓ_e is the load-bearing length of the anchor for shear:

$\ell_e = h_{ef}$ for anchors with a constant stiffness over the full length of embedded section, such as headed studs and post-installed anchors with one tubular shell over full length of the embedment depth;

$\ell_e = 2d_a$ for torque-controlled expansion anchors with a distance sleeve separated from expansion sleeve, and $\ell_e \leq 8d_a$ in all cases.

$$b. \quad V_b = 3.7 \lambda_a \sqrt{f'_c} (c_{a1})^{1.5} \quad (417.5.2.2a)$$

417.5.2.3 For cast-in headed studs, headed bolts, or hooked bolts that are continuously welded to steel attachments having a minimum thickness equal to the greater of 10 mm. and half of the anchor diameter, the basic concrete breakout strength in shear of a single

417.5.3 Concrete Pryout Strength of Anchor in Shear

417.5.3.1 The nominal pryout strength, V_{cp} for a single anchor or V_{cpg} for a group of anchors, shall not exceed:

a. For a single anchor;

$$V_{cp} = k_{cp}N_{cp} \quad (417.5.3.1a)$$

For cast-in, expansion, and undercut anchors, N_{cp} shall be taken as N_{cb} determined from Eq. 417.4.2.1a, and for adhesive anchors, N_{cp} shall be the lesser of N_a determined from Eq. 417.4.5.1a and N_{cb} determined from Eq. 417.4.2.1a.

b. For a group of anchors:

$$V_{cpg} = k_{cp}N_{cpg} \quad (417.5.3.1b)$$

For cast-in, expansion, and undercut anchors, N_{cpg} shall be taken as N_{cp} determined from Eq. 417.4.2.1b, and for adhesive anchors, N_{cpg} shall be the lesser of N_{ag} determined from Eq. 417.4.5.1b and N_{cp} determined from Eq. 417.4.2.1b.

In Eq. 417.5.3.1a and 417.5.3.1b, $k_{cp} = 1.0$ for $h_{ef} < 65$ mm.; and $k_{cp} = 2.0$ for $h_{ef} \geq 65$ mm.

417.6 Interaction of Tensile and Shear Forces

Unless determined in accordance with Section 417.3.1.3, anchors or groups of anchors that are subjected to both shear and axial loads shall be designed to satisfy the requirements of Sections 417.6.1 through 417.6.3. The values of ϕN_n and ϕV_n shall be the required strengths as determined from Section 417.3.1.1 or from Section 417.2.3.

417.6.1 If $V_{ua}/(\phi V_n) \leq 0.2$ for the governing strength in shear, then full strength in tension shall be permitted: $\phi N_n \geq N_{ua}$.

417.6.2 If $N_{ua}/(\phi N_n) \leq 0.2$ for the governing strength in tension, then full strength in shear shall be permitted: $\phi V_n \geq V_{ua}$.

417.6.3 If $V_{ua}/(\phi V_n) > 0.2$ for the governing strength in shear and $N_{ua}/(\phi N_n) > 0.2$ for the governing strength in tension, then:

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \quad (417.6.3)$$

417.7 Required Edge Distances, Spacings, and Thicknesses to Preclude Splitting Failure

Minimum spacings and edge distances for anchors and minimum thicknesses of members shall conform to Sections 417.7.1 through 417.7.6, unless supplementary reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with ACI 355.2 or ACI 355.4M shall be permitted.

417.7.1 Unless determined in accordance with Section 417.7.4, minimum center-to-center spacing of anchors shall be $4d_a$ for cast-in anchors that will not be torqued, and $6d_a$ for torqued cast-in anchors and post-installed anchors.

417.7.2 Unless determined in accordance with Section 417.7.4, minimum edge distances for cast-in anchors that will not be torqued shall be based on specified cover requirements for reinforcement in Section 420.6.1. For cast-in anchors that will be torqued, the minimum edge distances shall be $6d_a$.

417.7.3 Unless determined in accordance with Section 417.7.4, minimum edge distances for post-installed anchors shall be based on the greater of specified cover requirements for reinforcement in Section 420.6.1, or minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2 or ACI 355.4M, and shall not be less than twice the maximum aggregate size. In the absence of product-specific ACI 355.2 or ACI 355.4M test information, the minimum edge distance shall not be less than:

Adhesive anchors	$6d_a$
Undercut anchors	$6d_a$
Torque-controlled anchors	$8d_a$
Displacement-controlled anchors	$10d_a$

417.7.4 For anchors where installation does not produce a splitting force and that will not be torqued, if the edge distance or spacing is less than those specified in Sections 417.7.1 to 417.7.3, calculations shall be performed by substituting for d_a a smaller value d'_a that meets the requirements of Sections 417.7.1 to 417.7.3. Calculated forces applied to the anchor shall be limited to the values corresponding to an anchor having a diameter of d'_a .

417.7.5 Unless determined from tests in accordance with ACI 355.2, the value of h_{ef} for an expansion or undercut post-installed anchor shall not exceed the greater

of $2/3$ of the member thickness, h_a , and the member thickness minus 100 mm.

417.7.6 Unless determined from tension tests in accordance with ACI 355.2 or ACI 355.4M, the critical edge distance, c_{ac} , shall not be taken less than:

Adhesive anchors $2h_{ef}$

Undercut anchors $2.5h_{ef}$

Torque-controlled expansion anchors $4h_{ef}$

Displacement-controlled expansion anchors ... $4h_{ef}$

417.7.7 Construction documents shall specify use of anchors with a minimum edge distance as assumed in design.

417.8 Installation and Inspection of Anchors

417.8.1 Anchors shall be installed by qualified personnel in accordance with the construction documents and, where applicable, manufacturer's instructions. The construction documents shall require installation of post-installed adhesive anchors in accordance with the Manufacturer's Printed Installation Instructions (MPII). Installation of adhesive anchors shall be performed by personnel trained to install adhesive anchors.

417.8.2 Installation of anchors shall be inspected in accordance with Section 401.9 and the general building code. Adhesive anchors shall be also subject to Sections 417.8.2.1 through 417.8.2.4.

417.8.2.1 For adhesive anchors, the construction documents shall specify proof loading where required in accordance with ACI 355.4M. The construction documents shall also specify all parameters associated with the characteristic bond stress used for the design according to Section 417.4.5, including minimum age of concrete; concrete temperature range; moisture condition of concrete at time of installation; type of lightweight concrete, if applicable; and requirements for hole drilling and preparation.

417.8.2.2 Installation of adhesive anchors horizontally or upwardly inclined to support sustained tension loads shall be performed by personnel certified by an applicable certification program. Certification shall include written and performance tests in accordance with the ACI/CRSI Adhesive Anchor Installer Certification program, or equivalent.

417.8.2.3 The acceptability of certification other than the ACI/CRSI Adhesive Anchor Installer Certification shall be the responsibility of the licensed design professional.

417.8.2.4 Adhesive anchors installed in horizontal or upwardly inclined orientations to resist sustained tension loads shall be continuously inspected during installation by an inspector specially approved for that purpose by the building official. The special inspector shall furnish a report to the licensed design professional and building official that the work covered by the report has been performed and that the materials used and the installation procedures used conform with the approved construction documents and the Manufacturer's Printed Installation Instructions (MPII).

SECTION 418

EARTHQUAKE-RESISTANT STRUCTURES

418.1 Scope

418.1.1 This section shall apply to the design of non-prestressed and prestressed concrete structures assigned to seismic zone 4, including, where applicable:

- a. Structural systems designated as part of the seismic-force-resisting system, including diaphragms, moment frames, structural walls, and foundations;
- b. Members not designated as part of the seismic-force-resisting system but required to support other loads while undergoing deformations associated with earthquake effects.

418.1.2 Structures designed according to the provisions of this chapter are intended to resist earthquake motions through ductile inelastic response of selected members.

418.2 General

418.2.1 Structural Systems

418.2.1.1 All structures shall be assigned to a seismic zone in accordance with Section 404.4.6.1.

418.2.1.2 All members shall satisfy Sections 401 to 417 and 419 to 426. Structures assigned to seismic zones 4, or 2, also shall satisfy Sections 418.2.1.3 through 418.2.1.7, as applicable. Where Section 418 conflicts with other sections of this Code, Section 418 shall govern.

418.2.1.3 Structures assigned to seismic zone 2 shall satisfy Sections 418.2.2 and 418.2.3.

418.2.1.4 Structures assigned to seismic zone 4 shall satisfy Sections 418.2.2 through 418.2.8 and Sections 418.12 through 418.14.

418.2.1.5 Structural systems designated as part of the seismic-force-resisting system shall be restricted to those designated by the general building code, or determined by other authority having jurisdiction in areas without a legally adopted building code. Except for areas of low seismic risk, for which Section 418 does not apply, (a) through (h) shall be satisfied for each structural system designated as part of the seismic-force-resisting system, in addition to Sections 418.2.1.3 through 418.2.1.5:

- a. Ordinary moment frames shall satisfy Section 418.3;
- b. Ordinary reinforced concrete structural walls need not satisfy any detailing provisions in Section 418, unless required by Section 418.2.1.3 or 418.2.1.4.
- c. Intermediate moment frames shall satisfy Section 418.4;
- d. Intermediate precast walls shall satisfy Section 418.5;
- e. Special moment frames shall satisfy Sections 418.2.3 through 418.2.8 and 418.6 through 418.8;
- f. Special moment frames constructed using precast concrete shall satisfy Sections 418.2.3 through 418.2.8 and 418.9;
- g. Special structural walls shall satisfy Sections 418.2.3 through 418.2.8 and 418.10;
- h. Special structural walls constructed using precast concrete shall satisfy Sections 418.2.3 through 418.2.8 and 418.11.

418.2.1.6 A reinforced concrete structural system not satisfying this section shall be permitted if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable reinforced concrete structure satisfying this section..

418.2.2 Analysis and Proportioning of Structural Members

418.2.2.1 The interaction of all structural and nonstructural members that affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

418.2.2.2 Rigid members assumed not to be a part of the seismic force-resisting system shall be permitted provided their effect on the response of the system is considered in the structural design. Consequences of failure of structural and nonstructural members that are not a part of the seismic-force-resisting system shall be considered.

418.2.2.3 Structural members extending below the base of structure that are required to transmit forces resulting from earthquake effects to the foundation shall comply with the requirements of Section 418 that are consistent with the seismic-force-resisting system above the base of structure.

418.2.3 Anchoring to Concrete

418.2.3.1 Anchors resisting earthquake-induced forces in structures assigned to seismic zones 4 or 2 shall be in accordance with Section 417.2.3.

418.2.4 Strength Reduction Factors

418.2.4.1 Strength reduction factors shall be in accordance with Section 421.

418.2.5 Concrete in Special Moment Frames and Special Structural Walls

418.2.5.1 Specified compressive strength of concrete in special moment frames and special structural walls shall be in accordance with the special seismic systems requirements of Table 419.2.1.1.

418.2.6 Reinforcement in Special Moment Frames and Special Structural Walls

418.2.6.1 Reinforcement in special moment frames and special structural walls shall be in accordance with the special seismic systems requirements of Section 420.2.2.

418.2.7 Mechanical Splices in Special Moment Frames and Special Structural Walls

418.2.7.1 Mechanical splices shall be classified as (a) or (b):

- a. Type 1 – Mechanical splice conforming to Section 425.5.7;
- b. Type 2 – Mechanical splice conforming to Section 425.5.7 and capable of developing the specified tensile strength of the spliced bars.

418.2.7.2 Type 1 mechanical splices shall not be located within a distance equal to twice the member depth from the column or beam face for special moment frames or from critical sections where yielding of the reinforcement is likely to occur as a result of lateral displacements beyond the linear range of behavior. Type 2 mechanical splices shall be permitted at any location, except as noted in Section 418.9.2.1(c).

418.2.8 Welded Splices in Special Moment Frames and Special Structural Walls

418.2.8.1 Welded splices in reinforcement resisting earthquake-induced forces shall conform to Section 425.5.7 and shall not be located within a distance equal to twice the member depth from the column or beam face for special moment frames or from critical sections where

yielding of reinforcement is likely to occur as a result of lateral displacements beyond the linear range of behavior.

418.2.8.2 Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement required by design shall not be permitted.

418.3 Ordinary Moment Frames

418.3.1 Scope

418.3.1.1 This section shall apply to ordinary moment frames forming part of the seismic-force-resisting system.

418.3.2 Beams shall have at least two continuous bars at both top and bottom faces. Continuous bottom bars shall have area not less than one-fourth the maximum area of bottom bars along the span. These bars shall be anchored to develop f_y in tension at the face of support.

418.3.3 Columns having unsupported length $\ell_u \leq 5c_1$ shall have ϕV_n at least the lesser of (a) and (b):

- a. The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength;
- b. The maximum shear obtained from design load combinations that include E , with $\Omega_o E$ substituted for E .

418.4 Intermediate Moment Frames

418.4.1 Scope

418.4.1.1 This section shall apply to intermediate moment frames including two-way slabs without beams forming part of the seismic force-resisting system.

418.4.2 Beams

418.4.2.1 Beams shall have at least two continuous bars at both top and bottom faces. Continuous bottom bars shall have area not less than one-fourth the maximum area of bottom bars along the span. These bars shall be anchored to develop f_y in tension at the face of support.

418.4.2.2 The positive moment strength at the face of the joint shall be at least one-third the negative moment strength provided at that face of the joint. Neither the

negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

418.4.2.3 ϕV_n shall be at least the lesser of (a) and (b):

- The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity loads;
- The maximum shear obtained from design load combinations that include E , with E taken as twice that prescribed by the general building code.

418.4.2.4 At both ends of the beam, hoops shall be provided over a length of at least $2h$ measured from the face of the supporting member toward mid-span. The first hoop shall be located not more than 50 mm from the face of the supporting member. Spacing of hoops shall not exceed the smallest of (a) through (d):

- $d / 4$;
- Eight times the diameter of the smallest longitudinal bar enclosed;
- 24 times the diameter of the hoop bar;
- 300 mm.

418.4.2.5 Transverse reinforcement spacing shall not exceed $d/2$ throughout the length of the beam.

418.4.2.6 In beams having factored axial compressive force exceeding $A_g f'_c / 10$, transverse reinforcement required by Section 418.4.2.5 shall conform to Section 425.7.2.2 and either Section 425.7.2.3 or 425.7.2.4.

418.4.3 Columns

418.4.3.1 ϕV_n shall be at least the lesser of (a) and (b):

- The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength;

- The maximum shear obtained from factored load combinations that include E , with $\Omega_o E$ substituted for E .

418.4.3.2 Columns shall be spirally reinforced in accordance with Section 410 or shall be in accordance with Sections 418.4.3.3 through 418.4.3.5. Provisions of Section 418.4.3.6 shall apply to all columns supporting discontinuous stiff members.

418.4.3.3 At both ends of the column, hoops shall be provided at spacing s_o over a length ℓ_o measured from the joint face. Spacing s_o shall not exceed the smallest of (a) through (d):

- 8 times the diameter of the smallest longitudinal bar enclosed;
- 24 times the diameter of the hoop bar;
- One-half of the smallest cross-sectional dimension of the column;
- 300 mm.

Length ℓ_o shall not be less than the largest of (e), (f), and (g):

- One-sixth of the clear span of the column;
- Maximum cross-sectional dimension of the column;
- 450 mm.

418.4.3.4 The first hoop shall be located not more than $s_o/2$ from the joint face.

418.4.3.5 Outside of length ℓ_o , spacing of transverse reinforcement shall be in accordance with Section 410.7.6.5.2.

418.4.3.6 Columns supporting reactions from discontinuous stiff members, such as walls, shall be provided with transverse reinforcement at the spacing, s_o , in accordance with Section 418.4.3.3 over the full height beneath the level at which the discontinuity occurs if the portion of factored axial compressive force in these members related to earthquake effects exceeds $A_g f'_c / 10$. If design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $A_g f'_c / 10$ shall be increased to $A_g f'_c / 4$. Transverse reinforcement shall extend above and below the column in accordance with Section 418.7.5.6(b).

418.4.4 Joints

418.4.4.1 Beam-column joints shall have transverse reinforcement conforming to Section 415.

418.4.5 Two-Way Slabs without Beams

418.4.5.1 Factored slab moment at the support including earthquake effects, E , shall be calculated for load combinations given in Eq. 405.3.1e and 405.3.1g. Reinforcement to resist M_{sc} shall be placed within the column strip defined in Section 408.4.1.5.

418.4.5.2 Reinforcement placed within the effective width given in Section 408.4.2.3.3 shall be designed to resist $\gamma_f M_{sc}$. Effective slab width for exterior and corner connections shall not extend beyond the column face a distance greater than c_t measured perpendicular to the slab span.

418.4.5.3 At least one-half of the reinforcement in the column strip at the support shall be placed within the effective slab width given in Section 408.4.2.3.3.

418.4.5.4 At least one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span.

418.4.5.5 Continuous bottom reinforcement in the column strip shall be at least one-third of the top reinforcement at the support in the column strip.

418.4.5.6 At least one-half of all bottom middle strip reinforcement and all bottom column strip reinforcement at mid-span shall be continuous and shall develop f_y at the face of support as defined in Section 408.10.3.2.1.

418.4.5.7 At discontinuous edges of the slab, all top and bottom reinforcement at the support shall be developed at the face of support as defined in Section 408.10.3.2.1.

418.4.5.8 At the critical sections for columns defined in Section 422.6.4.1, two-way shear caused by factored gravity loads shall not exceed $0.4\phi V_c$, where V_c shall be calculated in accordance with Section 422.6.5. This requirement need not be satisfied if the slab satisfies Section 418.14.5.

418.5 Intermediate Precast Structural Walls

418.5.1 Scope

418.5.1.1 This section shall apply to intermediate precast structural walls forming part of the seismic-force-resisting system.

418.5.2 General

418.5.2.1 In connections between wall panels, or between wall panels and the foundation, yielding shall be restricted to steel elements or reinforcement.

418.5.2.2 For elements of the connection that are not designed to yield, the required strength shall be based on $1.5S_y$ of the yielding portion of the connection.

418.5.2.3 In structures assigned to seismic zone 4, wall piers shall be designed in accordance with Section 418.10.8 or 418.14.

418.6 Beams of Special Moment Frames

418.6.1 Scope

418.6.1.1 This section shall apply to beams of special moment frames that form part of the seismic-force-resisting system and are proportioned primarily to resist flexure and shear.

418.6.1.2 Beams of special moment frames shall frame into columns of special moment frames satisfying Section 418.7.

418.6.2 Dimensional Limits

418.6.2.1 Beams shall satisfy (a) through (c):

- Clear span, ℓ_n , shall be at least $4d$;
- Width, b_w , shall be at least the smaller of $0.3h$ and 250 mm;
- Projection of the beam width beyond the width of the supporting column on each side shall not exceed the smaller of c_2 and $0.75c_1$.

418.6.3 Longitudinal Reinforcement

418.6.3.1 Beams shall have at least two continuous bars at both top and bottom faces. At any section, for top as well as for bottom reinforcement, the amount of reinforcement shall be at least that required by Section 409.6.1.2 and the reinforcement ratio, ρ , shall not exceed 0.025.

418.6.3.2 Positive moment strength at joint face shall be at least one-half the negative moment strength provided at that face of the joint. Both the negative and the positive moment strength at any section along member length shall be at least one-fourth the maximum moment strength provided at face of either joint.

418.6.3.3 Lap splices of deformed longitudinal reinforcement shall be permitted if hoop or spiral reinforcement is provided over the lap length. Spacing of the transverse reinforcement enclosing the lap spliced bars shall not exceed the smaller of $d/4$ and 100 mm. Lap splices shall not be used in locations (a) through (c):

- Within the joints;
- Within a distance of twice the beam depth from the face of the joint;
- Within a distance of twice the beam depth from critical sections where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior.

418.6.3.4 Mechanical splices shall conform to Section 418.2.7 and welded splices shall conform to Section 418.2.8.

418.6.3.5 Unless used in a special moment frame as permitted by Section 418.9.2.3, prestressing shall satisfy (a) through (d):

- The average prestress, f_{pc} , calculated for an area equal to the smallest cross-sectional dimension of the beam multiplied by the perpendicular cross-sectional dimension shall not exceed the lesser of 3.5 MPa and $f'_c/10$;
- Prestressing steel shall be unbonded in potential plastic hinge regions, and the calculated strains in prestressing steel under the design displacement shall be less than one percent;
- Prestressing steel shall not contribute more than one-fourth of the positive or negative flexural strength at the critical section in a plastic hinge region and shall be anchored at or beyond the exterior face of the joint;
- Anchorage of post-tensioning tendons resisting earthquake-induced forces shall be capable of allowing tendons to withstand 50 cycles of loading, with prestressed reinforcement forces bounded by 40 and 85 percent of the specified tensile strength of the prestressing steel.

418.6.4 Transverse Reinforcement

418.6.4.1 Hoops shall be provided in the following regions of a beam:

- Over a length equal to twice the beam depth measured from the face of the supporting column toward mid-span, at both ends of the beam;
- Over lengths equal to twice the beam depth on both sides of a section where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior.

418.6.4.2 Where hoops are required, primary longitudinal reinforcing bars closest to the tension and compression faces shall have lateral support in accordance with Section 425.7.2.3 and 425.7.2.4. The spacing of transversely supported flexural reinforcing bars shall not exceed 350 mm. Skin reinforcement required by Section 409.7.2.3 need not be laterally supported.

418.6.4.3 Hoops in beams shall be permitted to be made up of two pieces of reinforcement: a stirrup having seismic hooks at both ends and closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90-degree hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the crossties are confined by a slab on only one side of the beam, the 90-degree hooks of the crossties shall be placed on that side.

418.6.4.4 The first hoop shall be located not more than 50 mm from the face of a supporting column. Spacing of the hoops shall not exceed the smallest of (a) through (c):

- $d/4$;
- Six times the diameter of the smallest primary flexural reinforcing bars excluding longitudinal skin reinforcement required by Section 409.7.2.3;
- 150 mm.

418.6.4.5 Where hoops are required, they shall be designed to resist shear according to Section 418.6.5.

418.6.4.6 Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than $d/2$ throughout the length of the beam.

418.6.4.7 In beams having factored axial compressive force exceeding $A_g f'_c/10$, hoops satisfying Sections 418.7.5.2 through 418.7.5.4 shall be provided along lengths given in Section 418.6.4.1. Along the remaining length, hoops satisfying Section 418.7.5.2 shall have spacing, s , not exceeding the smaller of six times the diameter of the smallest longitudinal beam bars and 150 mm. Where concrete cover over transverse reinforcement exceeds 100 mm, additional transverse reinforcement

having cover not exceeding 100 mm and spacing not exceeding 300 mm shall be provided.

418.6.5 Shear Strength

418.6.5.1 Design Forces

The design shear force, V_e , shall be calculated from consideration of the forces on the portion of the beam between faces of the joints. It shall be assumed that moments of opposite sign corresponding to probable flexural strength, M_{pr} , act at the joint faces and that the beam is loaded with the factored tributary gravity load along its span.

418.6.5.2 Transverse Reinforcement

Transverse reinforcement over the lengths identified in Section 418.6.4.1 shall be designed to resist shear assuming $V_c = 0$ when both (a) and (b) occur:

- The earthquake-induced shear force calculated in accordance with Section 418.6.5.1 represents at least one-half of the maximum required shear strength within those lengths;
- The factored axial compressive force, P_u , including earthquake effects is less than $A_g f'_c / 20$.

418.7 Columns of Special Moment Frames

418.7.1 Scope

418.7.1.1 This section shall apply to columns of special moment frames that form part of the seismic-force-resisting system and are proportioned primarily to resist flexure, shear, and axial forces.

418.7.2 Dimensional Limits

418.7.2.1 Columns shall satisfy (a) and (b):

- The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall be at least 300 mm;
- The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall be at least 0.4.

418.7.3 Minimum Flexural Strength of Columns

418.7.3.1 Columns shall satisfy Section 418.7.3.2 or 418.7.3.3:

418.7.3.2 The flexural strengths of the columns shall satisfy

$$\Sigma M_{nc} \geq (6/5) \Sigma M_{nb} \quad (418.7.3.2)$$

where

ΣM_{nc} = sum of nominal flexural strengths of columns framing into the joint, evaluated at the faces of the joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.

ΣM_{nb} = sum of nominal flexural strengths of the beams framing into the joint, evaluated at the faces of the joint. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width defined in accordance with Section 406.3.2 shall be assumed to contribute to M_{nb} if the slab reinforcement is developed at the critical section for flexure.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Equation 418.7.3.2 shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

418.7.3.3 If Section 418.7.3.2 is not satisfied at a joint, the lateral strength and stiffness of the columns framing into that joint shall be ignored when calculating strength and stiffness of the structure. These columns shall conform to Section 418.14.

418.7.4 Longitudinal Reinforcement

418.7.4.1 Area of longitudinal reinforcement, A_{st} , shall be at least $0.01A_g$ and shall not exceed $0.06A_g$

418.7.4.2 In columns with circular hoops, there shall be at least six longitudinal bars.

418.7.4.3 Mechanical splices shall conform to Section 418.2.7 and welded splices shall conform to Section 418.2.8. Lap splices shall be permitted only within the center half of the member length, shall be designed as tension lap splices, and shall be enclosed within transverse reinforcement in accordance with Sections 418.7.5.2 and 418.7.5.3.

418.7.5 Transverse Reinforcement

418.7.5.1 Transverse reinforcement required in Sections 418.7.5.2 through 418.7.5.4 shall be provided over a length ℓ_o from each joint face and on both sides of any section where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior. Length ℓ_o shall be at least the largest of (a) through (c):

- The depth of the column at the joint face or at the section where flexural yielding is likely to occur;
- One-sixth of the clear span of the column;
- 450 mm.

418.7.5.2 Transverse reinforcement shall be in accordance with (a) through (f):

- Transverse reinforcement shall comprise either single or overlapping spirals, circular hoops, or rectilinear hoops with or without crossties;
- Bends of rectilinear hoops and crossties shall engage peripheral longitudinal reinforcing bars;
- Crossties of the same or smaller bar size as the hoops shall be permitted, subject to the limitation of Section 425.7.2.2. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement and around the perimeter of the cross section;
- Where rectilinear hoops or crossties are used, they shall provide lateral support to longitudinal reinforcement in accordance with Sections 425.7.2.2 and 425.7.2.3;
- Reinforcement shall be arranged such that the spacing h_x of longitudinal bars laterally supported by the corner of a crosstie or hoop leg shall not exceed 350 mm around the perimeter of the column;
- Where $P_u > 0.3A_g f'_c$ or $f'_c > 70$ MPa in columns with rectilinear hoops, every longitudinal bar or bundle of bars around the perimeter of the column core shall have lateral support provided by the corner of a hoop or by a seismic hook, and the value of h_x shall not exceed 200 mm. P_u shall be the largest value in compression consistent with factored load combinations including E .

418.7.5.3 Spacing of transverse reinforcement shall not exceed the smallest of (a) through (c):

- One-fourth of the minimum column dimension;

- Six times the diameter of the smallest longitudinal bar;

- s_o as calculated by

$$s_o = 100 + \left(\frac{350 - h_x}{3} \right) \quad (418.7.5.3)$$

The value of s_o from Eq. 418.7.5.3 shall not exceed 150 mm. and need not be taken less than 100 mm.

418.7.5.4 Amount of transverse reinforcement shall be in accordance with Table 418.7.5.4.

The concrete strength factor, k_f , and confinement effectiveness factor, k_n , are calculated according to Eq. 418.7.5.4a and 418.7.5.4b.

$$a. \quad k_f = \frac{f'_c}{175} + 0.6 \geq 1.0 \quad (418.7.5.4a)$$

$$b. \quad k_n = \frac{n_l}{n_l - 2} \quad (418.7.5.4b)$$

where n_l is the number of longitudinal bars or bar bundles around the perimeter of a column core with rectilinear hoops that are laterally supported by the corner of hoops or by seismic hooks.

418.7.5.5 Beyond the length ℓ_o given in Section 418.7.5.1, the column shall contain spiral or hoop reinforcement satisfying Sections 425.7.2 through 425.7.4 with spacing, s , not exceeding the smaller of six times the diameter of the smallest longitudinal column bars and 150 mm., unless a larger amount of transverse reinforcement is required by Section 418.7.4.3 or 418.7.6.

418.7.5.6 Columns supporting reactions from discontinued stiff members, such as walls, shall satisfy (a) and (b):

- Transverse reinforcement required by Sections 418.7.5.2 through 418.7.5.4 shall be provided over the full height at all levels beneath the discontinuity if the factored axial compressive force in these columns, related to earthquake effect, exceeds $A_g f'_c / 10$. Where design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $A_g f'_c / 10$ shall be increased to $A_g f'_c / 4$;
- Transverse reinforcement shall extend into the discontinued member at least ℓ_d of the largest longitudinal column bar, where ℓ_d is in accordance

with Section 418.8.5. Where the lower end of the column terminates on a wall, the required transverse reinforcement shall extend into the wall at least ℓ_d of

the largest longitudinal column bar at the point of termination. Where the column terminates on a footing

Table 418.7.5.4
Transverse Reinforcement for Columns of Special Moment Frames

Transverse Reinforcement	Conditions	Applicable Expressions	
$\frac{A_{sh}}{sb_c}$ for rectilinear hoop	$P_u \leq 0.30A_g f'_c$ and $f'_c \leq 70$ MPa	Greater of (a) and (b)	$0.3 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (a)
	$P_u > 0.30A_g f'_c$ or $f'_c > 70$ MPa	Greatest of (a), (b), and (c)	$0.09 \frac{f'_c}{f_{yt}}$ (b) $0.2k_f k_n \frac{P_u}{f_{yt} A_{ch}}$ (c)
ρ_s for spiral or circular hoop	$P_u \leq 0.30A_g f'_c$ and $f'_c \leq 70$ MPa	Greater of (d) and (e)	$0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (d)
	$P_u > 0.30A_g f'_c$ or $f'_c > 70$ MPa	Greatest of (d), (e), and (f)	$0.12 \frac{f'_c}{f_{yt}}$ (e)
			$0.35k_f k_n \frac{P_u}{f_{yt} A_{ch}}$ (f)

or mat, the required transverse reinforcement shall extend at least 300 mm into the footing or mat.

418.7.5.7 If the concrete cover outside the confining transverse reinforcement required by Sections 418.7.5.1, 418.7.5.5, and 418.7.5.6 exceeds 100 mm, additional transverse reinforcement having cover not exceeding 100 mm, and spacing not exceeding 300 mm shall be provided.

418.7.6 Shear Strength

418.7.6.1 Design Forces

418.7.6.1.1 The design shear force V_e shall be calculated from considering the maximum forces that can be generated at the faces of the joints at each end of the column. These joint forces shall be calculated using the maximum probable flexural strengths, M_{pr} , at each end of the column associated with the range of factored axial forces, P_u , acting on the column. The column shears need not exceed those calculated from joint strengths based on M_{pr} of the beams framing into the joint. In no case shall V_e be less than the factored shear calculated by analysis of the structure.

418.7.6.2 Transverse Reinforcement

418.7.6.2.1 Transverse reinforcement over the lengths ℓ_o , given in Section 418.7.5.1, shall be designed to resist shear assuming $V_c = 0$ when both (a) and (b) occur:

- The earthquake-induced shear force, calculated in accordance with Section 418.7.6.1, is at least one-half of the maximum required shear strength within ℓ_o ;
- The factored axial compressive force, P_u , including earthquake effects is less than $A_g f'_c / 20$.

418.8 Joints of Special Moment Frames

418.8.1 Scope

418.8.1.1 This section shall apply to beam-column joints of special moment frames forming part of the seismic-force-resisting system.

418.8.2 General

418.8.2.1 Forces in longitudinal beam reinforcement at the joint face shall be calculated assuming that the stress in the flexural tensile reinforcement is $1.25f_y$.

418.8.2.2 Beam longitudinal reinforcement terminated in a column shall extend to the far face of the confined column core and shall be developed in tension in

accordance with Section 418.8.5 and in compression in accordance with Section 425.4.9.

418.8.2.3 Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall be at least 20 times the diameter of the largest longitudinal beam bar for normal-weight concrete or 26 times the diameter of the largest longitudinal bar for lightweight concrete.

418.8.2.4 Depth h of the joint shall not be less than one-half of depth h of any beam framing into the joint and generating joint shear as part of the seismic-force-resisting system.

418.8.3 Transverse Reinforcement

418.8.3.1 Joint transverse reinforcement shall satisfy Sections 418.7.5.2, 418.7.5.3, 418.7.5.4, and 418.7.5.7, except as permitted in Section 418.8.3.2.

418.8.3.2 Where beams frame into all four sides of the joint and where each beam width is at least three-fourths the column width, the amount of reinforcement required by Section 418.7.5.4 shall be permitted to be reduced by one-half, and the spacing required by Section 418.7.5.3 shall be permitted to be increased to 150 mm within the overall depth h of the shallowest framing beam.

418.8.3.3 Longitudinal beam reinforcement outside the column core shall be confined by transverse reinforcement passing through the column that satisfies spacing requirements of Section 418.6.4.4, and requirements of Sections 418.6.4.2, and 418.6.4.3, if such confinement is not provided by a beam framing into the joint.

418.8.3.4 Where beam negative moment reinforcement is provided by headed deformed bars that terminate in the joint, the column shall extend above the top of the joint a distance at least the depth h of the joint. Alternatively, the beam reinforcement shall be enclosed by additional vertical joint reinforcement providing equivalent confinement to the top face of the joint.

418.8.4 Shear Strength

418.8.4.1 V_n of the joint shall be in accordance with Table 418.8.4.1.

Table 418.8.4.1
Nominal Joint Shear Strength V_n

Joint configuration	V_n
For joints confined by beams on all four faces ^[1]	$1.7\lambda\sqrt{f'_c}A_j$ ^[2]
For joints confined by beams on three faces or on two opposite faces ^[1]	$1.2\lambda\sqrt{f'_c}A_j$ ^[2]
For other cases	$1.0\lambda\sqrt{f'_c}A_j$ ^[2]

^[1] Refer to Section 418.8.4.2.

^[2] λ shall be 0.75 for lightweight concrete and 1.0 for normal-weight concrete. A_j is given in Section 418.8.4.3.

418.8.4.2 In Table 418.8.4.1, a joint face is considered to be confined by a beam if the beam width is at least three-quarters of the effective joint width. Extensions of beams at least one overall beam depth h beyond the joint face are considered adequate for confining that joint face. Extensions of beams shall satisfy Sections 418.6.2(b), 418.6.3.1, 418.6.4.2, 418.6.4.3, and 418.6.4.4.

418.8.4.3 Effective cross-sectional area within a joint, A_j , shall be calculated from joint depth times effective joint width. Joint depth shall be the overall depth of the column, h . Effective joint width shall be the overall width of the column, except where a beam frames into a wider column, effective joint width shall not exceed the smaller of (a) and (b):

- Beam width plus joint depth;
- Twice the smaller perpendicular distance from longitudinal axis of beam to column side.

418.8.5 Development Length of Bars in Tension

418.8.5.1 For bar sizes 10 mm ϕ through 36 mm ϕ terminating in a standard hook, ℓ_{dh} shall be calculated by Eq. 418.8.5.1, but ℓ_{dh} shall be at least the larger of $8d_b$ and 150 mm. for normal-weight concrete and at least the larger of $10d_b$ and 190 mm. for lightweight concrete.

$$\ell_{dh} = f_y d_b / (5.4\lambda\sqrt{f'_c}) \quad (418.8.5.1)$$

The value of λ shall be 0.75 for lightweight and 1.0 for normal-weight concrete.

The hook shall be located within the confined core of a column or of a boundary element, with the hook bent into the joint.

418.8.5.2 For headed deformed bars satisfying Section 420.2.1.6, development in tension shall be in accordance

with Section 425.4.4, except clear spacing between bars shall be permitted to be at least $3d_b$ or greater.

418.8.5.3 For bar sizes 10 mm ϕ through 36 mm ϕ , ℓ_d , the development length in tension for a straight bar, shall be at least the larger of (a) and (b):

- a. 2.5 times the length in accordance with Section 418.8.5.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 300 mm;
- b. 3.25 times the length in accordance with Section 418.8.5.1 if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm.

418.8.5.4 Straight bars terminated at a joint shall pass through the confined core of a column or a boundary element. Any portion of ℓ_d not within the confined core shall be increased by a factor of 1.6.

418.8.5.5 If epoxy-coated reinforcement is used, the development lengths in Sections 418.8.5.1, 418.8.5.3, and 418.8.5.4 shall be multiplied by applicable factors in Section 425.4.2.4 or 425.4.3.2.

418.9 Special Moment Frames Constructed Using Precast Concrete

418.9.1 Scope

418.9.1.1 This section shall apply to special moment frames constructed using precast concrete forming part of the seismic-force resisting system.

418.9.2 General

418.9.2.1 Special moment frames with ductile connections constructed using precast concrete shall satisfy (a) through (c):

- a. Requirements of Sections 418.6 through 418.8 for special moment frames constructed with cast-in-place concrete;
- b. V_n for connections calculated according to Section 422.9 shall be at least $2V_e$, where V_e is in accordance with Section 418.6.5.1 or 418.7.6.1;
- c. Mechanical splices of beam reinforcement shall be located not closer than $h/2$ from the joint face and shall satisfy Section 418.2.7.

418.9.2.2 Special moment frames with strong connections constructed using precast concrete shall satisfy (a) through (e):

- a. Requirements of Sections 418.6 through 418.8 for special moment frames constructed with cast-in-place concrete;
- b. Provision Section 418.6.2.1(a) shall apply to segments between locations where flexural yielding is intended to occur due to design displacements;
- c. Design strength of the strong connection, ϕS_n , shall be at least S_e ;
- d. Primary longitudinal reinforcement shall be made continuous across connections and shall be developed outside both the strong connection and the plastic hinge region;
- e. For column-to-column connections, ϕS_n shall be at least $1.4S_e$, ϕM_n shall be at least $0.4M_{pr}$ for the column within the story height, and ϕV_n shall be at least V_e in accordance with Section 418.7.6.1.

418.9.2.3 Special moment frames constructed using precast concrete and not satisfying Section 418.9.2.1 or 418.9.2.2 shall satisfy (a) through (c):

- a. ACI 374.1;
- b. Details and materials used in the test specimens shall be representative of those used in the structure;
- c. The design procedure used to proportion the test specimens shall define the mechanism by which the frame resists gravity and earthquake effects, and shall establish acceptance values for sustaining that mechanism. Portions of the mechanism that deviate from Code requirements shall be contained in the test specimens and shall be tested to determine upper bounds for acceptance values.

418.10 Special Structural Walls

418.10.1 Scope

418.10.1.1 This section shall apply to special structural walls and all components of special structural walls including coupling beams and wall piers forming part of the seismic-force-resisting system.

418.10.1.2 Special structural walls constructed using precast concrete shall be in accordance with Section 418.11 in addition to Section 418.10.

418.10.2 Reinforcement

418.10.2.1 The distributed web reinforcement ratios, ρ_l and ρ_t , for structural walls shall be at least 0.0025,

except that if V_u does not exceed $0.083A_{cv}\lambda\sqrt{f'_c}$, ρ_l and ρ_t shall be permitted to be reduced to the values in Section 411.6. Reinforcement spacing each way in structural walls shall not exceed 450 mm. Reinforcement contributing to V_n shall be continuous and shall be distributed across the shear plane.

418.10.2.2 At least two curtains of reinforcement shall be used in a wall if $V_u > 0.17A_{cv}\lambda\sqrt{f'_c}$ or $h_w/\ell_w \geq 2.0$, in which h_w and ℓ_w refer to height and length of entire wall, respectively.

418.10.2.3 Reinforcement in structural walls shall be developed or spliced for f_y in tension in accordance with Sections 425.4, 425.5, and (a) through (c):

- Longitudinal reinforcement shall extend beyond the point at which it is no longer required to resist flexure by least $0.8\ell_w$, except at the top of a wall;
- At locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements, development lengths of longitudinal reinforcement shall be 1.25 times the values calculated for f_y in tension;
- Mechanical splices of reinforcement shall conform to Section 418.2.7 and welded splices of reinforcement shall conform to Section 418.2.8.

418.10.3 Design Forces

V_u shall be obtained from the lateral load analysis in accordance with the factored load combinations.

418.10.4 Shear Strength

418.10.4.1 V_n of structural walls shall not exceed:

$$V_u = A_{cv}(\alpha_c\lambda\sqrt{f'_c} + \rho_t f_y) \quad (418.10.4.1)$$

where the coefficient α_c is 0.25 for $h_w/\ell_w \leq 1.5$, is 0.17 for $h_w/\ell_w \geq 2.0$, and varies linearly between 0.25 and 0.17 for h_w/ℓ_w between 1.5 and 2.0.

418.10.4.2 In Section 418.10.4.1, the value of ratio h_w/ℓ_w used to calculate V_n for segments of a wall shall be the larger of the ratios for the entire wall and the segment of wall considered.

418.10.4.3 Walls shall have distributed shear reinforcement in two orthogonal directions in the plane of the wall. If h_w/ℓ_w does not exceed 2.0, reinforcement ratio ρ_t shall be at least the reinforcement ratio ρ_l .

418.10.4.4 For all vertical wall segments sharing a common lateral force, V_n shall not be taken larger than $0.66A_{cv}\lambda\sqrt{f'_c}$, where A_{cv} is the gross area of concrete bounded by web thickness and length of section. For any one of the individual vertical wall segments, V_n shall not be taken larger than $0.83A_{cw}\lambda\sqrt{f'_c}$, where A_{cw} is the area of concrete section of the individual vertical wall segment considered.

418.10.4.5 For horizontal wall segments and coupling beams, V_n shall not be taken larger than $0.83A_{cw}\lambda\sqrt{f'_c}$, where A_{cw} is the area of concrete section of a horizontal wall segment or coupling beam.

418.10.5 Design for Flexure and Axial Force

418.10.5.1 Structural walls and portions of such walls subject to combined flexure and axial loads shall be designed in accordance with Section 422.4. Concrete and developed longitudinal reinforcement within effective flange widths, boundary elements, and the wall web shall be considered effective. The effects of openings shall be considered.

418.10.5.2 Unless a more detailed analysis is performed, effective flange widths of flanged sections shall extend from the face of the web a distance equal to the lesser of one-half the distance to an adjacent wall web and 25 percent of the total wall height.

418.10.6 Boundary Elements of Special Structural Walls

418.10.6.1 The need for special boundary elements at the edges of structural walls shall be evaluated in accordance with Section 418.10.6.2 or 418.10.6.3. The requirements of Sections 418.10.6.4 and 418.10.6.5 shall also be satisfied.

418.10.6.2 Walls or wall piers with $h_w/\ell_w \geq 2.0$ that are effectively continuous from the base of structure to top of wall and are designed to have a single critical section for flexure and axial loads shall satisfy (a) and (b) or shall be designed by Section 418.10.6.3:

- Compression zones shall be reinforced with special boundary elements where

$$c \geq \frac{\ell_w}{600(\delta_u/h_w)} \quad (418.10.6.2)$$

and c corresponds to the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the direction of the

design displacement δ_u . Ratio δ_u/h_w shall not be taken less than **0.005**.

- b. Where special boundary elements are required by (a), the special boundary element transverse reinforcement shall extend vertically above and below the critical section at least the larger of ℓ_w and $M_u/4V_u$, except as permitted in Section 418.10.6.4(g).

418.10.6.3 Structural walls not designed in accordance with Section 418.10.6.2 shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to load combinations including earthquake effects, E , exceeds **0.2** f'_c . The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than **0.15** f'_c . Stresses shall be calculated for the factored loads using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as given in Section 418.10.5.2 shall be used.

418.10.6.4 Where special boundary elements are required by Section 418.10.6.2 or 418.10.6.3, (a) through (h) shall be satisfied:

- The boundary element shall extend horizontally from the extreme compression fiber a distance be at least the larger of $c - 0.1\ell_w$ and $c/2$, where c is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with δ_u ;
- Width of flexural compression zone b over the horizontal distance calculated by Section 418.10.6.4(a), including flange if present, shall be at least $h_u/16$;
- For walls or wall piers with $h_w/\ell_w \geq 2.0$ that are effectively continuous from the base of structure to top of wall, designed to have a single critical section for flexure and axial loads, and with $c/\ell_w \geq 3/8$, width of the flexural compression zone b over the length calculated in Section 418.10.6.4(a) shall be greater than or equal to 300 mm;
- In flanged sections, the boundary element shall include the effective flange width in compression and shall extend at least 300 mm. into the web;
- The boundary element transverse reinforcement shall satisfy Sections 418.7.5.2(a) through (e) and Section 418.7.5.3, except the value h_x in Section 418.7.5.2 shall not exceed the lesser of 350 mm and two-thirds of the boundary element thickness, and the transverse

reinforcement spacing limit of Section 418.7.5.3(a) shall be one-third of the least dimension of the boundary element;

- f. The amount of transverse reinforcement shall be in accordance with Table 418.10.6.4(f).

Table 418.10.6.4(f)
Transverse Reinforcement for
Special Boundary Elements

Transverse reinforcement	Applicable expressions		
$\frac{A_{sh}}{sb_c}$ For rectilinear hoop	Greater of	$0.3 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$	(a)
		$0.09 \frac{f'_c}{f_{yt}}$	(b)
ρ_s For spiral or circular hoop	Greater of	$0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$	(c)
		$0.12 \frac{f'_c}{f_{yt}}$	(d)

- Where the critical section occurs at the wall base, the boundary element transverse reinforcement at the wall base shall extend into the support at least ℓ_d , in accordance with Section 418.10.2.3, of the largest longitudinal reinforcement in the special boundary element. Where the special boundary element terminates on a footing, mat, or pile cap, special boundary element transverse reinforcement shall extend at least 300 mm into the footing, mat, or pile cap, unless a greater extension is required by Section 408.13.2.3;
- Horizontal reinforcement in the wall web shall extend to within 150 mm of the end of the wall. Reinforcement shall be anchored to develop f_y within the confined core of the boundary element using standard hooks or heads. Where the confined boundary element has sufficient length to develop the horizontal web reinforcement, and $A_s f_y / s$ of the horizontal web reinforcement does not exceed $A_s f_{yt} / s$ of the boundary element transverse reinforcement parallel to the horizontal web reinforcement, it shall be permitted to terminate the horizontal web reinforcement without a standard hook or head.

418.10.6.5 Where special boundary elements are not required by Section 418.10.6.2 or 418.10.6.3, (a) and (b) shall be satisfied:

- If the longitudinal reinforcement ratio at the wall boundary exceeds **2.8**/ f_y , boundary transverse

reinforcement shall satisfy Section 418.7.5.2(a) through (c) over the distance calculated in accordance with Section 418.10.6.4(a). The longitudinal spacing of transverse reinforcement at the wall boundary shall not exceed the lesser of 200 mm and $8d_b$ of the smallest primary flexural reinforcing bars, except the spacing shall not exceed the lesser of 150 mm and $6d_b$ within a distance equal to the greater of ℓ_w and $M_u/4V_u$ above and below critical sections where yielding of longitudinal reinforcement is likely to occur as a result of in elastic lateral displacements;

- b. Except where V_u in the plane of the wall is less than $0.083A_{cv}\lambda\sqrt{f'_c}$, horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement, shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

418.10.7 Coupling Beams

418.10.7.1 Coupling beams with $(\ell_n/h) \geq 4$ shall satisfy the requirements of Section 418.6, with the wall boundary interpreted as being a column. The provisions of Sections 418.6.2.1(b) and 418.6.2.1(c) need not be satisfied if it can be shown by analysis that the beam has adequate lateral stability.

418.10.7.2 Coupling beams with $(\ell_n/h) < 2$ and with V_u exceeding $0.33\lambda\sqrt{f'_c}A_{cw}$ shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the mid-span, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load-carrying ability of the structure, the egress from the structure, or the integrity of nonstructural components and their connections to the structure.

418.10.7.3 Coupling beams not governed by Section 418.10.7.1 or 418.10.7.2 shall be permitted to be reinforced either with two intersecting groups of diagonally placed bars symmetrical about the mid-span or according to Sections 418.6.3 through 418.6.5, with the wall boundary interpreted as being a column.

418.10.7.4 Coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the mid-span shall satisfy (a), (b), and either (c) or (d), and the requirements of Section 409.9 need not be satisfied:

- a. V_n shall be calculated by

$$V_n = 2A_{vd}f_y \sin \alpha \leq 0.83\sqrt{f'_c}A_{cw} \quad (418.10.7.4)$$

where α is the angle between the diagonal bars and the longitudinal axis of the coupling beam;

- b. Each group of diagonal bars shall consist of a minimum of four bars provided in two or more layers. The diagonal bars shall be embedded into the wall at least 1.25 times the development length for f_y in tension;
- c. Each group of diagonal bars shall be enclosed by rectilinear transverse reinforcement having out-to-out dimensions of at least $b_w/2$ in the direction parallel to b_w and $b_w/5$ along the other sides, where b_w is the web width of the coupling beam. The transverse reinforcement shall be in accordance with Section 418.7.5.2(a) through (e), with A_{sh} not less than the greater of (i) and (ii):

i) $0.09sb_c \frac{f'_c}{f_{yt}}$

ii) $0.3sb_c \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$

For the purpose of calculating A_g , the concrete cover in Section 420.6.1 shall be assumed on all four sides of each group of diagonal bars. The transverse reinforcement shall have spacing measured parallel to the diagonal bars satisfying Section 418.7.5.3(c) and not exceeding $6d_b$ of the smallest diagonal bars, and shall have spacing of crossties or legs of hoops measured perpendicular to the diagonal bars not exceeding 350 mm. The transverse reinforcement shall continue through the intersection of the diagonal bars. At the intersection, it is permitted to modify the arrangement of the transverse reinforcement provided the spacing and volume ratio requirements are satisfied. Additional longitudinal and transverse reinforcement shall be distributed around the beam perimeter with total area in each direction of at least $0.002b_ws$ and spacing not exceeding 300 mm.

- d. Transverse reinforcement shall be provided for the entire beam cross section in accordance with Section 418.7.5.2(a) through (e) with A_{sh} not less than the greater of (i) and (ii):

i) $0.09sb_c \frac{f'_c}{f_{yt}}$

$$\text{ii) } 0.3sb_c \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$$

Longitudinal spacing of transverse reinforcement shall not exceed the smaller of 150 mm and $6d_b$ of the smallest diagonal bars. Spacing of crossties or legs of hoops both vertically and horizontally in the plane of the beam cross section shall not exceed 200 mm. Each crosstie and each hoop leg shall engage a longitudinal bar of equal or greater diameter. It shall be permitted to configure hoops as specified in Section 418.6.4.3.

418.10.8 Wall Piers

418.10.8.1 Wall piers shall satisfy the special moment frame requirements for columns of Sections 418.7.4, 418.7.5, and 418.7.6, with joint faces taken as the top and bottom of the clear height of the wall pier. Alternatively, wall piers with $(\ell_w/b_w) > 2.5$ shall satisfy (a) through (f):

- Design shear force shall be calculated in accordance with Section 418.7.6.1 with joint faces taken as the top and bottom of the clear height of the wall pier. If the general building code includes provisions to account for overstrength of the seismic-force-resisting system, the design shear force need not exceed Ω_o times the factored shear calculated by analysis of the structure for earthquake load effects;
- V_n and distributed shear reinforcement shall satisfy Section 418.10.4;
- Transverse reinforcement shall be hoops except it shall be permitted to use single-leg horizontal reinforcement parallel to ℓ_w where only one curtain of distributed shear reinforcement is provided. Single-leg horizontal reinforcement shall have 180-degree bends at each end that engage wall pier boundary longitudinal reinforcement;
- Vertical spacing of transverse reinforcement shall not exceed 150 mm;
- Transverse reinforcement shall extend at least 300 mm. above and below the clear height of the wall pier;
- Special boundary elements shall be provided if required by Section 418.10.6.3.

418.10.8.2 For wall piers at the edge of a wall, horizontal reinforcement shall be provided in adjacent wall segments above and below the wall pier and be designed to transfer

the design shear force from the wall pier into the adjacent wall segments.

418.10.9 Construction Joints

418.10.9.1 Construction joints in structural walls shall be specified according to Section 426.5.6, and contact surfaces shall be roughened consistent with the condition selected in Section 422.9.4.2.

418.10.10 Discontinuous Walls

418.10.10.1 Columns supporting discontinuous structural walls shall be reinforced in accordance with Section 418.7.5.6.

418.11 Special Structural Walls Constructed Using Precast Concrete

418.11.1 Scope

418.11.1.1 This section shall apply to special structural walls constructed using precast concrete forming part of the seismic-force resisting system.

418.11.2 General

418.11.2.1 Special structural walls constructed using precast concrete shall satisfy Section 418.10 and 418.5.2.

418.11.2.2 Special structural walls constructed using precast concrete and unbonded post-tensioning tendons and not satisfying the requirements of Section 418.11.2.1 are permitted provided they satisfy the requirements of ACI ITG 5.1.

418.12 Diaphragms and Trusses

418.12.1 Scope

418.12.1.1 This section shall apply to diaphragms and collectors forming part of the seismic-force-resisting system in structures assigned to seismic zone 4.

418.12.1.2 Section 418.12.11 shall apply to structural trusses forming part of the seismic-force-resisting system in structures assigned to seismic zone 4.

418.12.2 Design Forces

418.12.2.1 The earthquake design forces for diaphragms shall be obtained from the general building code using the applicable provisions and load combinations.

$$\text{ii) } 0.3sb_c \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$$

Longitudinal spacing of transverse reinforcement shall not exceed the smaller of 150 mm and $6d_b$ of the smallest diagonal bars. Spacing of crossties or legs of hoops both vertically and horizontally in the plane of the beam cross section shall not exceed 200 mm. Each crosstie and each hoop leg shall engage a longitudinal bar of equal or greater diameter. It shall be permitted to configure hoops as specified in Section 418.6.4.3.

418.10.8 Wall Piers

418.10.8.1 Wall piers shall satisfy the special moment frame requirements for columns of Sections 418.7.4, 418.7.5, and 418.7.6, with joint faces taken as the top and bottom of the clear height of the wall pier. Alternatively, wall piers with $(\ell_w/b_w) > 2.5$ shall satisfy (a) through (f):

- Design shear force shall be calculated in accordance with Section 418.7.6.1 with joint faces taken as the top and bottom of the clear height of the wall pier. If the general building code includes provisions to account for overstrength of the seismic-force-resisting system, the design shear force need not exceed Ω_o times the factored shear calculated by analysis of the structure for earthquake load effects;
- V_n and distributed shear reinforcement shall satisfy Section 418.10.4;
- Transverse reinforcement shall be hoops except it shall be permitted to use single-leg horizontal reinforcement parallel to ℓ_w where only one curtain of distributed shear reinforcement is provided. Single-leg horizontal reinforcement shall have 180-degree bends at each end that engage wall pier boundary longitudinal reinforcement;
- Vertical spacing of transverse reinforcement shall not exceed 150 mm;
- Transverse reinforcement shall extend at least 300 mm. above and below the clear height of the wall pier;
- Special boundary elements shall be provided if required by Section 418.10.6.3.

418.10.8.2 For wall piers at the edge of a wall, horizontal reinforcement shall be provided in adjacent wall segments above and below the wall pier and be designed to transfer

the design shear force from the wall pier into the adjacent wall segments.

418.10.9 Construction Joints

418.10.9.1 Construction joints in structural walls shall be specified according to Section 426.5.6, and contact surfaces shall be roughened consistent with the condition selected in Section 422.9.4.2.

418.10.10 Discontinuous Walls

418.10.10.1 Columns supporting discontinuous structural walls shall be reinforced in accordance with Section 418.7.5.6.

418.11 Special Structural Walls Constructed Using Precast Concrete

418.11.1 Scope

418.11.1.1 This section shall apply to special structural walls constructed using precast concrete forming part of the seismic-force resisting system.

418.11.2 General

418.11.2.1 Special structural walls constructed using precast concrete shall satisfy Section 418.10 and 418.5.2.

418.11.2.2 Special structural walls constructed using precast concrete and unbonded post-tensioning tendons and not satisfying the requirements of Section 418.11.2.1 are permitted provided they satisfy the requirements of ACI ITG 5.1.

418.12 Diaphragms and Trusses

418.12.1 Scope

418.12.1.1 This section shall apply to diaphragms and collectors forming part of the seismic-force-resisting system in structures assigned to seismic zone 4.

418.12.1.2 Section 418.12.1.1 shall apply to structural trusses forming part of the seismic-force-resisting system in structures assigned to seismic zone 4.

418.12.2 Design Forces

418.12.2.1 The earthquake design forces for diaphragms shall be obtained from the general building code using the applicable provisions and load combinations.

418.12.3 Seismic Load Path

418.12.3.1 All diaphragms and their connections shall be designed and detailed to provide for transfer of forces to collector elements and to the vertical elements of the seismic-force-resisting system.

418.12.3.2 Elements of a structural diaphragm system that are subjected primarily to axial forces and used to transfer diaphragm shear or flexural forces around openings or other discontinuities shall satisfy the requirements for collectors in Sections 418.12.7.5 and 418.12.7.6.

418.12.4 Cast-In-Place Composite Topping Slab Diaphragms

418.12.4.1 A cast-in-place composite topping slab on a precast floor or roof shall be permitted as a structural diaphragm, provided the cast-in-place topping slab is reinforced and the surface of the previously hardened concrete on which the topping slab is placed is clean, free of laitance, and intentionally roughened.

418.12.5 Cast-in-Place Non-Composite Topping Slab Diaphragms

418.12.5.1 A cast-in-place non-composite topping on a precast floor or roof shall be permitted as a structural diaphragm, provided the cast-in-place topping slab acting alone is designed and detailed to resist the design earthquake forces.

418.12.6 Minimum Thickness of Diaphragms

418.12.6.1 Concrete slabs and composite topping slabs serving as diaphragms used to transmit earthquake forces shall be at least 50 mm thick. Topping slabs placed over precast floor or roof elements, acting as diaphragms and not relying on composite action with the precast elements to resist the design earthquake forces, shall be at least 65 mm thick.

418.12.7 Reinforcement

418.12.7.1 The minimum reinforcement ratio for diaphragms shall be in conformance with Section 424.4. Except for post-tensioned slabs, reinforcement spacing each way in floor or roof systems shall not exceed 450 mm. Where welded wire reinforcement is used as the distributed reinforcement to resist shear in topping slabs placed over precast floor and roof elements, the wires parallel to the joints between the precast elements shall be spaced not less than 250 mm on center. Reinforcement provided for shear strength shall be continuous and shall be distributed uniformly across the shear plane.

418.12.7.2 Bonded tendons used as reinforcement to resist collector forces, diaphragm shear, or flexural tension shall be designed such that the stress due to design earthquake forces does not exceed 420 MPa. Pre-compression from unbonded tendons shall be permitted to resist diaphragm design forces if a seismic load path is provided.

418.12.7.3 All reinforcement used to resist collector forces, diaphragm shear, or flexural tension shall be developed or spliced for f_y in tension.

418.12.7.4 Type 2 splices are required where mechanical splices are used to transfer forces between the diaphragm and the vertical elements of the seismic-force-resisting system.

418.12.7.5 Collector elements with compressive stresses exceeding $0.2f'_c$ at any section shall have transverse reinforcement satisfying Sections 418.7.5.2(a) through 418.7.5.2(e) and 418.7.5.3, except the spacing limit of Section 418.7.5.3(a) shall be one-third of the least dimension of the collector. The amount of transverse reinforcement shall be in accordance with Table 418.12.7.5. The specified transverse reinforcement is permitted to be discontinued at a section where the calculated compressive stress is less than $0.15f'_c$.

If design forces have been amplified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $0.2f'_c$ shall be increased to $0.5f'_c$, and the limit of $0.15f'_c$ shall be increased to $0.4f'_c$.

Table 418.12.7.5
Transverse Reinforcement for Collector Elements

Transverse reinforcement	Applicable expressions	
$\frac{A_{sh}}{sb_c}$ for rectilinear hoop	$0.09 \frac{f'_c}{f_{yt}}$	(a)
ρ_s for spiral or circular hoop	Greater of	$0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (b)
		$0.12 \frac{f'_c}{f_{yt}}$ (c)

418.12.7.6 Longitudinal reinforcement detailing for collector elements at splices and anchorage zones shall satisfy (a) or (b):

- Center-to-center spacing of at least three longitudinal bar diameters, but not less than 40 mm, and concrete

clear cover of at least two and one-half longitudinal bar diameters, but not less than 50 mm;

- b. Area of transverse reinforcement, providing A_v at least the greater of $0.062\sqrt{f'_c}(b_w s/f_{yt})$ and $0.35b_w s/f_{yt}$, except as required in Section 418.12.7.5

418.12.8 Flexural Strength

418.12.8.1 Diaphragms and portions of diaphragms shall be designed for flexure in accordance with Section 412. The effects of openings shall be considered.

418.12.9 Shear Strength

418.12.9.1 V_n of diaphragms shall not exceed:

$$V_n = A_{cv}(0.17\lambda\sqrt{f'_c} + \rho_t f_y) \quad (418.12.9.1)$$

For cast-in-place topping slab diaphragms on precast floor or roof members, A_{cv} shall be calculated using only the thickness of topping slab for non-composite topping slab diaphragms and the combined thickness of cast-in-place and precast elements for composite topping slab diaphragms. For composite topping slab diaphragms, the value of f'_c used to calculate V_n shall not exceed the smaller of f'_c for the precast members and f'_c for the topping slab.

418.12.9.2 V_n of diaphragms shall not exceed $0.66A_{cv}\sqrt{f'_c}$.

418.12.9.3 Above joints between precast elements in non-composite and composite cast-in-place topping slab diaphragms, V_n shall not exceed:

$$V_n = A_{vf} f_y \mu \quad (418.12.9.3)$$

where A_{vf} is the total area of shear friction reinforcement within the topping slab, including both distributed and boundary reinforcement, that is oriented perpendicular to joints in the precast system and coefficient of friction, μ , is **1.0**, where λ is given in Section 419.2.4. At least one-half of A_{vf} shall be uniformly distributed along the length of the potential shear plane. The area of distributed reinforcement in the topping slab shall satisfy Section 424.4.3.2 in each direction.

418.12.9.4 Above joints between precast elements in non-composite and composite cast-in-place topping slab diaphragms, V_n shall not exceed the limits in Section 422.9.4.4, where A_c is calculated using only the thickness of the topping slab.

418.12.10 Construction Joints

418.12.10.1 Construction joints in diaphragms shall be specified according to Section 426.5.6, and contact surfaces shall be roughened consistent with the condition selected in Section 422.9.4.2.

418.12.11 Structural Trusses

418.12.11.1 Structural truss elements with compressive stresses exceeding $0.2f'_c$ at any section shall have transverse reinforcement, in accordance with Sections 418.7.5.2, 418.7.5.3, 418.7.5.7, and Table 418.12.11.1, over the length of the element.

Table 418.12.11.1

Transverse Reinforcement for Structural Trusses

Transverse reinforcement	Applicable expressions		
$\frac{A_{sh}}{sb_c}$ for rectilinear hoop	Greater of	$0.3 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$	(a)
		$0.09 \frac{f'_c}{f_{yt}}$	(b)
ρ_s for spiral or circular hoop	Greater of	$0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$	(c)
		$0.12 \frac{f'_c}{f_{yt}}$	(d)

418.12.11.2 All continuous reinforcement in structural truss elements shall be developed or spliced for f_y in tension.

418.13 Foundations

418.13.1 Scope

418.13.1.1 This section shall apply to foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground in structures assigned to seismic zone 4.

418.13.1.2 The provisions in this section for piles, drilled piers, caissons, and slabs-on-ground shall supplement other applicable Code design and construction criteria, including Sections 401.4.5 and 401.4.6.

418.13.2 Footings, Foundation Mats, and Pile Caps

418.13.2.1 Longitudinal reinforcement of columns and structural walls resisting forces induced by earthquake effects shall extend into the footing, mat, or pile cap, and shall be fully developed for tension at the interface.

418.13.2.2 Columns designed assuming fixed-end conditions at the foundation shall comply with Section 418.13.2.1 and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90-degree hooks near the bottom of the foundation with the free end of the bars oriented toward the center of the column.

418.13.2.3 Columns or boundary elements of special structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with Sections 418.7.5.2 through 418.7.5.4 provided below the top of the footing. This reinforcement shall extend into the footing, mat, or pile cap a length equal to the development length, calculated for f_y in tension, of the column or boundary element longitudinal reinforcement.

418.13.2.4 Where earthquake effects create uplift forces in boundary elements of special structural walls or columns, flexural reinforcement shall be provided in the top of the footing, mat or pile cap to resist actions resulting from the factored load combinations, and shall be at least that required by Section 407.6.1 or 409.6.1.

418.13.2.5 Structural plain concrete in footings and basement walls shall be in accordance with Section 414.1.4.

418.13.3 Grade Beams and Slabs-on-Ground

418.13.3.1 Grade beams designed to act as horizontal ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supported column or anchored within the pile cap or footing at all discontinuities.

418.13.3.2 Grade beams designed to act as horizontal ties between pile caps or footings shall be sized such that the smallest cross-sectional dimension shall be at least equal to the clear spacing between connected columns divided by 20, but need not exceed 450 mm. Closed ties shall be provided at a spacing not to exceed the lesser of one-half the smallest orthogonal cross-sectional dimension and 300 mm.

418.13.3.3 Grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the seismic-force-resisting system shall be in accordance with Section 418.6.

418.13.3.4 Slabs-on-ground that resist earthquake forces from walls or columns that are part of the seismic-force-resisting system shall be designed as diaphragms in accordance with Section 418.12. The construction documents shall clearly indicate that the slab-on-ground is

a structural diaphragm and part of the seismic-force-resisting system.

418.13.4 Piles, Piers, and Caissons

418.13.4.1 Piles, piers, or caissons resisting tension loads shall have continuous longitudinal reinforcement over the length resisting design tension forces. The longitudinal reinforcement shall be detailed to transfer tension forces within the pile cap to supported structural members.

418.13.4.2 Where tension forces induced by earthquake effects are transferred between pile cap or mat foundation and precast pile by reinforcing bars grouted or post-installed in the top of the pile, the grouting system shall have been demonstrated by test to develop at least $1.25f_y$ of the bar.

418.13.4.3 Piles, piers, or caissons shall have transverse reinforcement in accordance with Sections 418.7.5.2(a) through (e), 418.7.5.3 and 418.7.5.4 excluding requirements of (c) and (f) of Table 418.7.5.4 at locations (a) and (b):

- a. At the top of the member for at least five times the member cross-sectional dimension, and at least 1.8 m below the bottom of the pile cap;
- b. For the portion of piles in soil that is not capable of providing lateral support, or in air and water, along the entire unsupported length plus the length required in (a).

418.13.4.4 For precast concrete driven piles, the length of transverse reinforcement provided shall be sufficient to account for potential variations in the elevation of pile tips.

418.13.4.5 Concrete piles, piers, or caissons in foundations supporting one- and two-story stud bearing wall construction are exempt from the transverse reinforcement requirements of Sections 418.13.4.3 and 418.13.4.4.

418.13.4.6 Pile caps incorporating batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns. The slenderness effects of batter piles shall be considered for the portion of the piles in soil that is not capable of providing lateral support, or in air or water.

418.14 Members Not Designated as Part of the Seismic-Force-Resisting System

418.14.1 Scope

418.14.1.1 This section shall apply to members not designated as part of the seismic-force-resisting system in structures assigned to seismic zone 4.

418.14.2 Design Actions

418.14.2.1 Members not designated as part of the seismic force-resisting system shall be evaluated for gravity load combinations of $(1.2D + 1.0L + 0.2S)$ or $0.9D$, whichever is critical, acting simultaneously with the design displacement δ_u . The load factor on the live load, L , shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 4.8 kPa.

418.14.3 Cast-In-Place Beams, Columns, and Joints

418.14.3.1 Cast-in-place beams and columns shall be detailed in accordance with Section 418.14.3.2 or 418.14.3.3 depending on the magnitude of moments and shears induced in those members when subjected to the design displacement δ_u . If effects of δ_u are not explicitly checked, the provisions of Section 418.14.3.3 shall be satisfied.

418.14.3.2 Where the induced moments and shears do not exceed the design moment and shear strength of the frame member, (a) through (c) shall be satisfied:

- Beams shall satisfy Section 418.6.3.1. Transverse reinforcement shall be provided throughout the length of the beam at spacing not to exceed $d/2$. Where factored axial force exceeds $A_g f'_c/10$, transverse reinforcement shall be hoops satisfying Section 418.7.5.2 at spacing so, according to Section 418.14.3.2(b);
- Columns shall satisfy Sections 418.7.4.1, 418.7.5.2, and 418.7.6. The maximum longitudinal spacing of hoops shall be s_o for the full column length. Spacing s_o shall not exceed the smaller of six diameters of the smallest longitudinal bar enclosed and 150 mm;
- Columns with factored gravity axial forces exceeding $0.35P_o$ shall satisfy Sections 418.14.3.2(b) and 418.7.5.7. The amount of transverse reinforcement provided shall be one-half of that required by Section 418.7.5.4 and spacing shall not exceed s_o for the full column length.

418.14.3.3 Where the induced moments or shears exceed ϕM_n or ϕV_n of the frame member, or if induced moments or shears are not calculated, (a) through (d) shall be satisfied:

- Materials, mechanical splices, and welded splices shall satisfy the requirements for special moment frames in Sections 418.2.5 through 418.2.8;
- Beams shall satisfy Sections 418.14.3.2(a) and 418.6.5;
- Columns shall satisfy Sections 418.7.4, 418.7.5, and 418.7.6;
- Joints shall satisfy Section 418.8.3.1.

418.14.4 Precast Beams and Columns

418.14.4.1 Precast concrete frame members assumed not to contribute to lateral resistance, including their connections, shall satisfy (a) through (d):

- Requirements of Section 418.14.3;
- Ties specified in Section 418.14.3.2(b) over the entire column height, including the depth of the beams;
- Structural integrity reinforcement, in accordance with Section 404.10;
- Bearing length at the support of a beam shall be at least 50 mm longer than determined from Section 416.2.6.

418.14.5 Slab-Column Connections

418.14.5.1 For slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the requirements of Section 408.7.6 or 408.7.7 shall be provided at any slab critical section defined in Section 422.6.4.1 if $\Delta_x/h_{sx} \geq 0.035 - (1/20)(v_{ug}/\phi v_c)$. Required slab shear reinforcement shall provide $v_s \geq 0.29\sqrt{f'_c}$ at the slab critical section and shall extend at least four times the slab thickness from the face of the support adjacent to the slab critical section. The shear reinforcement requirements of this provision shall not apply if $\Delta_x/h_{sx} \leq 0.005$.

The value of (Δ_x/h_{sx}) shall be taken as the greater of the values of the adjacent storeys above and below the slab-column connection. v_c shall be calculated in accordance with Section 422.6.5. v_{ug} is the factored shear stress on the slab critical section for two-way action due to gravity loads without moment transfer.

418.14.6 Wall Piers

418.14.6.1 Wall piers not designated as part of the seismic-force resisting system shall satisfy the requirements of Section 418.10.8. Where the general building code includes provisions to account for overstrength of the seismic-force-resisting system, it shall be permitted to calculate the design shear force as Ω_o times the shear induced under design displacements, δ_u .

SECTION 419

CONCRETE: DESIGN AND DURABILITY REQUIREMENTS

419.1 Scope

419.1.1 This section shall apply to concrete, including:

- a. Properties to be used for design;
- b. Durability requirements

419.1.2 This section shall apply to durability requirements for grout used for bonded tendons in accordance with Section 419.4.

419.2 Concrete Design Properties**419.2.1 Specified Compressive Strength**

419.2.1.1 The value of f'_c shall be specified in construction documents and shall be in accordance with (a) through (c):

- a. Limits in Table 419.2.1.1;
- b. Durability requirements in Table 419.3.2.1;
- c. Structural strength requirements.

Table 419.2.1.1 Limits for f'_c

Application	Concrete	Minimum f'_c , MPa	Maximum f'_c , MPa
General	Normal-weight and lightweight	17	None
Special moment frames and special structural walls	Normal-weight	21	None
	Lightweight	21	35 ^[1]

^[1] The limit is permitted to be exceeded where demonstrated by experimental evidence that members made with lightweight concrete provide strength and toughness equal to or exceeding those of comparable members made with normal weight concrete of the same strength.

419.2.1.2 The specified compressive strength shall be used for proportioning of concrete mixtures in Section 426.4.3 and for testing and acceptance of concrete in Section 426.12.3.

419.2.1.3 Unless otherwise specified, f'_c shall be based on 28-day tests. If other than 28 days, test age for f'_c shall be indicated in the construction documents.

419.2.2 Modulus of Elasticity

419.2.2.1 Modulus of elasticity, E_c , for concrete shall be permitted to be calculated as (a) or (b):

- a. For values of w_c between 1440 and 2560 kg/m³

$$E_c = w_c^{1.5} 0.043 \sqrt{f'_c} \quad (\text{in MPa}) \quad (419.2.2.1.a)$$

- b. For normal weight concrete

$$E_c = 4700 \sqrt{f'_c} \quad (\text{in MPa}) \quad (419.2.2.1.b)$$

419.2.3 Modulus of Rupture

419.2.3.1 Modulus of rupture, f_r , for concrete shall be calculated by:

$$f_r = 0.62 \lambda \sqrt{f'_c} \quad (419.2.3.1)$$

where the value of λ is in accordance with Section 419.2.4.

419.2.4 Lightweight Concrete

419.2.4.1 To account for the properties of lightweight concrete, a modification factor λ is used as a multiplier of $\sqrt{f'_c}$ in all applicable provisions of this Code.

419.2.4.2 The value of λ shall be based on the composition of the aggregate in the concrete mixture in accordance with Table 419.2.4.2 or as permitted in Section 419.2.4.3.

Table 419.2.4.2
Modification Factor

Concrete	Composition of aggregates	λ
All-lightweight	Fine: ASTM C330M Coarse: ASTM C330M	0.75
Lightweight, fine blend	Fine: Combination of ASTM C330M and C33M Coarse: ASTM C330M	0.75 to 0.85 ^[1]
Sand-lightweight	Fine: ASTM C33M Coarse: ASTM C330M	0.85
Sand-lightweight, coarse blend	Fine: ASTM C33M Coarse: Combination of ASTM C330M and C33M	0.85 to 1.00 ^[2]
Normal-weight	Fine or Coarse: ASTM C33M	1.00

^[1] Linear interpolation from 0.75 to 0.85 is permitted based on the absolute volume of normal weight fine aggregate as a fraction of the total absolute volume of fine aggregate.^[2] Linear interpolation from 0.85 to 1.00 is permitted based on the absolute volume of normal-weight coarse aggregate as a fraction of the total absolute volume of coarse aggregate.

419.2.4.3 If the measured average splitting tensile strength of lightweight concrete, f_{ct} , is used to calculate λ , laboratory tests shall be conducted in accordance with ASTM C330M to establish the value of f_{ct} and the corresponding value of f_{cm} and λ shall be calculated by:

$$\lambda = \frac{f_{ct}}{0.56 \sqrt{f_{cm}}} \leq 1.0 \quad (419.2.4.3)$$

The concrete mixture tested in order to calculate λ shall be representative of that to be used in the Work.

419.3 Concrete Durability Requirements

419.3.1 Exposure Categories and Classes

419.3.1.1 The licensed design professional shall assign exposure classes in accordance with the severity of the anticipated exposure of members for each exposure category in Table 419.3.1.1.

Table 419.3.1.1
Exposure Categories and Classes

Category	Class	Condition	
Freezing and thawing (F)	F0	Concrete not exposed to freezing-and-thawing cycles	
	F1	Concrete exposed to freezing-and-thawing cycles with limited exposure to water	
	F2	Concrete exposed to freezing-and-thawing cycles with frequent exposure to water	
	F3	Concrete exposed to freezing-and-thawing cycles with frequent exposure to water and exposure to deicing chemicals	
		Water-soluble sulfate (SO_4^{2-}) in soil, percent by mass ^[1]	Dissolved Sulfate (SO_4^{2-}) in water, ppm ^[2]
	S0	$SO_4^{2-} < 0.10$	$SO_4^{2-} < 150$
	S1	$0.10 \leq SO_4^{2-} < 0.20$	$150 \leq SO_4^{2-} < 1500$ or seawater
	S2	$0.20 \leq SO_4^{2-} \leq 2.00$	$1500 \leq SO_4^{2-} \leq 10,000$
	S3	$SO_4^{2-} > 2.00$	$SO_4^{2-} > 10,000$
In contact with water (W)	W0	Concrete dry in service. Concrete in contact with water and low permeability is not required	
	W1	Concrete in contact with water and low permeability is required	
Corrosion protection of reinforcement (C)	C0	Concrete dry or protected from moisture	
	C1	Concrete exposed to moisture but not to an external source of chlorides	
	C2	Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources	

^[1] Percent sulfate by mass in soil shall be determined by ASTM C1580

^[2] Concentration of dissolved sulfates in water in ppm shall be determined by ASTM D516 or ASTM D4130

419.3.2 Requirements for Concrete Mixtures

419.3.2.1 Based on the exposure classes assigned from Table 419.3.1.1, concrete mixtures shall conform to the most restrictive requirements in Table 419.3.2.1.

Table 419.3.2.1 Requirements for Concrete by Exposure Class

Exposure Class	Maximum w/cm ^[1]	Minimum f'_c , MPa	Additional requirements			
			Air content			Limits on cementitious materials
F0	N/A	17	N/A			N/A
F1	0.55	24	Table 419.3.3.1			N/A
F2	0.45	31	Table 419.3.3.1			N/A
F3	0.40 ^[2]	35 ^[2]	Table 419.3.3.1			Section 426.4.2.2(b)
			Cementitious materials ^[3] - Types			Calcium chloride admixture
			ASTM C150M	ASTM C595M	ASTM C1157M	
S0	N/A	17	No Type restriction	No Type restriction	No Type restriction	No restriction
S1	0.50	28	II ^{[4][5]}	Types IP,IS, or ITwith (MS) designation	MS	No restriction
S2	0.45	31	V ^[5]	Types IP,IS, or ITwith (HS) designation	HS	Not permitted
S3	0.45	31	V plus pozzolan or slag cement ^[6]	Types IP,IS, or ITwith (HS) designation plus pozzolan or slag cement ^[6]	HS plus pozzolan or slag cement ^[6]	Not permitted
W0	N/A	17	None			
W1	0.50	28	None			
			Maximum water-soluble chloride ion (Cl-) content in concrete, percent by weight of cement ^[7]			Additional provisions
			Non-prestressed concrete		Prestressed concrete	
C0	N/A	17	1.00		0.06	None
C1	N/A	17	0.30		0.06	
C2	0.40	35	0.15		0.06	Concrete cover ^[8]

^[1] The maximum w/cm limits in Table 419.3.2.1 do not apply to lightweight concrete.

^[2] For plain concrete, the maximum w/cm shall be 0.45 and the minimum f'_c shall be 31.0 MPa.

^[3] Alternative combinations of cementitious materials to those listed in Table 419.3.2.1 are permitted when tested for sulfate resistance and meeting the criteria in Section 426.4.2.2(c).

^[4] For seawater exposure, other types of portland cements with tricalcium aluminate (C_3A) contents up to 10 percent are permitted if the w/cm does not exceed 0.40.

^[5] Other available types of cement such as Type I or Type III are permitted in Exposure Classes S1 or S2 if the C_3A contents are less than 8 percent for Exposure Class S1 or less than 5 percent for Exposure Class S2.

^[6] The amount of the specific source of the pozzolan or slag cement to be used shall be at least the amount that has been determined by service record to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slag cement to be used shall be at least the amount tested in accordance with ASTM C1012 and meeting the criteria in Section 426.4.2.2(c).

^[7] Water-soluble chloride ion content that is contributed from the ingredients including water, aggregates, cementitious materials, and admixtures shall be determined on the concrete mixture by ASTM C1218M at age between 28 and 42 days.

^[8] Concrete cover shall be in accordance with Section 420.6.

419.3.3 Additional Requirements for Freezing and Thawing Exposure

419.3.3.1 Normal-weight and lightweight concrete subject to freezing and thawing Exposure Classes F1, F2, or F3 shall be air entrained. Except as permitted in Section 419.3.3.3, air content shall conform to Table 419.3.3.1.

Table 419.3.3.1
Total Air Content for Concrete Exposed to Cycles of Freezing and Thawing

Nominal maximum aggregate size, mm.	Target air content, percent	
	F1	F2 and F3
10	6	7.5
12	5.5	7
20	5	6
25	4.5	6
40	4.5	5.5
50	4	5
75	3.5	4.5

419.3.3.2 Concrete shall be sampled in accordance with ASTM C172, and air content shall be measured in accordance with ASTM C231M or ASTM C173M.

419.3.3.3 For f'_c exceeding 35 MPa, reduction of air content indicated in Table 419.3.3.1 by 1.0 percentage point is permitted.

419.3.3.4 The maximum percentage of pozzolans, including fly ash and silica fume, and slag cement in concrete assigned to Exposure Class F3, shall be in accordance with Section 426.4.2.2(b).

419.3.4 Alternative Combinations of Cementitious Materials for Sulfate Exposure

419.3.4.1 Alternative combinations of cementitious materials to those listed in Section 419.3.2 are permitted when tested for sulfate resistance. Testing and acceptance criteria shall conform to Table 426.4.2.2(c).

419.4 Grout Durability Requirements

419.4.1 Water-soluble chloride ion content of grout for bonded tendons shall not exceed 0.06 percent when tested in accordance with ASTM C1218M, measured by mass of chloride ion to mass of cement.

SECTION 420 STEEL REINFORCEMENT PROPERTIES, DURABILITY, AND EMBEDMENTS

420.1 Scope

420.1.1 This section shall apply to steel reinforcement, and shall govern (a) through (c):

- Material properties;
- Properties to be used for design;
- Durability requirements, including minimum specified cover requirements.

420.1.2 Provisions of Section 420.7 shall apply to embedments.

420.2 Non-Prestressed Bars and Wires

420.2.1 Material Properties

420.2.1.1 Non-prestressed bars and wires shall be deformed, except plain bars or wires are permitted for use in spirals.

420.2.1.2 Yield strength of non-prestressed bars and wires shall be determined by either (a) or (b):

- The offset method, using an offset of 0.2 percent in accordance with ASTM A370;
- The yield point by the halt-of-force method, provided the non-prestressed bar or wire has a sharp-knead or well-defined type of yield point.

420.2.1.3 Deformed bars shall conform to (a), (b), (c), (d), or (e).

- ASTM A615M – carbon steel;
- ASTM A706M – low-alloy steel;
- ASTM A996M – axle steel and rail steel; bars from rail steel shall be Type R;
- ASTM A955M – stainless steel;
- ASTM A1035M – low-carbon chromium steel.

420.2.1.4 Plain bars for spiral reinforcement shall conform to ASTM A615M, A706M, A955M, or A1035M.

420.2.1.5 Welded deformed bar mats shall conform to ASTM A184M. Deformed bars used in welded deformed bar mats shall conform to ASTM A615M or A706M.

420.2.1.6 Headed deformed bars shall conform to ASTM A970M, including Annex A1 requirements for Class HA head dimensions.

420.2.1.7 Deformed wire, plain wire, welded deformed wire reinforcement, and welded plain wire reinforcement shall conform to (a) or (b), except that yield strength shall be determined in accordance with 420.2.1.2:

- a. A1064M – carbon steel;
- b. A1022M – stainless steel.

420.2.1.7.1 Deformed wire sizes MD25 through MD200 shall be permitted.

420.2.1.7.2 Deformed wire sizes larger than MD200 shall be permitted in welded wire reinforcement if treated as plain wire for calculation of development and splice lengths in accordance with Sections 425.4.7 and 425.5.4, respectively.

420.2.1.7.3 Except as permitted for welded wire reinforcement used as stirrups in accordance with Section 425.7.1, spacing of welded intersections in welded wire reinforcement in direction of calculated stress shall not exceed (a) or (b):

- a. 400 mm for welded deformed wire reinforcement;
- b. 300 mm for welded plain wire reinforcement.

420.2.2 Design Properties

420.2.2.1 For non-prestressed bars and wires, the stress below f_y shall be E_s times steel strain. For strains greater than that corresponding to f_y , stress shall be considered independent of strain and equal to f_y .

420.2.2.2 Modulus of elasticity, E_s , for nonprestressed bars and wires shall be permitted to be taken as 200,000 MPa.

420.2.2.3 Yield strength for non-prestressed bars and wires shall be based on the specified grade of reinforcement and shall not exceed the values given in Section 420.2.2.4 for the associated applications.

420.2.2.4 Types of non-prestressed bars and wires to be specified for particular structural applications shall be in accordance with Table 420.2.2.4(a) for deformed reinforcement and Table 420.2.2.4(b) for plain reinforcement.

420.2.2.5 Deformed non-prestressed longitudinal reinforcement resisting earthquake-induced moment, axial force, or both, in special moment frames, special structural walls, and all components of special structural walls including coupling beams and wall piers shall be in accordance with (a) or (b):

- a) ASTM A706M, Grade 420;
- b) ASTM A615M Grade 280 reinforcement if (i) and are satisfied and ASTM A615M Grade 420 reinforcement if (i) through (iii) are satisfied.
 - i. actual yield strength based on mill tests does not exceed f_y by more than 125 MPa;
 - ii. ratio of the actual tensile strength to the actual yield strength is at least 1.25;
 - iii. minimum elongation in 200 mm shall be at least 14 percent for bar sizes 10 mm ϕ through 20 mm ϕ , at least 12 percent for bar sizes 25 mm ϕ through 36 mm ϕ , and at least 10 percent for bar sizes 40 mm ϕ and 58 mm ϕ .

420.3 Prestressing Strands, Wires, and Bars

420.3.1 Material Properties

420.3.1.1 Except as required in Section 420.3.1.3 for special moment frames and special structural walls, prestressing reinforcement shall conform to (a), (b), (c), or (d):

- a. ASTM A416M – strand;
- b. ASTM A421M – wire;
- c. ASTM A421M – low-relaxation wire including Supplementary Requirement S1 “Low-Relaxation Wire and Relaxation Testing”;
- d. ASTM A722M – high-strength bar.

420.3.1.2 Prestressing strands, wires, and bars not listed in ASTM A416M, A421M, or A722M are permitted provided they conform to minimum requirements of these specifications and are shown by test or analysis not to impair the performance of the member.

420.3.1.3 Prestressing reinforcement resisting earthquake-induced moment and axial force, or both, in special moment frames, special structural walls, and all components of special structural walls including coupling

beams and wall piers, cast using precast concrete shall comply with ASTM A416M or A722M.

Table 420.2.2.4(a)
Non-Prestressed Deformed Reinforcement

Usage	Application	Maximum value of f_y or f_{yt} permitted for design calculations, MPa	Applicable ASTM Specification			
			Deformed Bars	Deformed Wires	Welded wire reinforcement	Welded deformed bar mats
Flexure, axial force, and shrinkage and temperature	Special seismic systems	420	Refer to Sect. 420.2.2.5	Not permitted	Not permitted	Not permitted
	Other	550	A615M, A706M, A955M, A996M	A1064M, A1022M	A1064M, A1022M	A184M[1]
Lateral support of longitudinal bars or concrete confinement	Special seismic systems	700	A615M, A706M, A955M, A996M, A1035M	A1064M, A1022M	A1064M ^[2] , A1022M ^[2]	Not permitted
	Spirals	700	A615M, A706M, A955M, A996M, A1035M	A1064M, A1022M	Not permitted	Not permitted
	Others	550	A615M, A706M, A955M, A996M	A1064M, A1022M	A1064M, A1022M	Not permitted
Shear	Special Seismic Systems	420	A615M, A706M, A955M, A996M	A1064M, A1022M	Not permitted	Not permitted
	Spirals	420	A615M, A706M, A955M, A996M	A1064M, A1022M	A1064M, A1022M	Not permitted
	Shear friction	420	A615M, A706M, A955M, A996M	A1064M, A1022M	A1064M and A1022M welded plain wire	Not permitted
	Stirrups, ties, hoops	550	A615M, A706M, A955M, A996M	A1064M, A1022M	A1064M and A1022M welded deformed wire	Not permitted
Torsion	Longitudinal and transverse	420	A615M, A706M, A955M, A996M	A1064M, A1022M	A1064M, A1022M	Not permitted

^[1] Welded deformed bar mats shall be permitted to be assembled using A615M or A706M deformed bars.

^[2] ASTM A1064M and A1022M are not permitted in special seismic systems where the weld is required to resist stresses in response to confinement, lateral support of longitudinal bars, shear, or other actions.

Table 420.2.2.4(b) Non-Prestressed Plain Spiral Reinforcement

Usage	Application	Maximum value of f_y or f_{yt} permitted for design calculations, MPa	Applicable ASTM Specification	
			Plain Bars	Plain Wires
Lateral support of longitudinal bars or Concrete confinement	Spirals in special seismic systems	700	A615M, A706M, A955M, A1035M	A1064M, A1022M
	Spirals	700	A615M, A706M, A955M, A1035M	A1064M, A1022M
Shear	Spirals	420	A615M, A706M, A955M, A1035M	A1064M, A1022M
Torsion in Non-prestressed beams	Spirals	420	A615M, A706M, A955M, A1035M	A1064M, A1022M

Table 420.3.2.2 Prestressing Strands, Wires, and Bars

Type	Maximum value of f_{pu} permitted for design calculations, MPa	Applicable ASTM Specification
Strand (stress-relieved and low-relaxation)	1860	A416M
Wire (stress-relieved and low-relaxation)	1725	A421M
		A421M Including Supplementary Requirement S1 "Low-Relaxation Wire and Relaxation Testing"
High-strength bar	1035	A422M

420.3.2 Design Properties

420.3.2.1 Modulus of elasticity, E_p , for prestressing reinforcement shall be determined from tests or as reported by the manufacturer.

420.3.2.2 Tensile strength, f_{pu} , shall be based on the specified grade or type of prestressing reinforcement and shall not exceed the values given in Table 420.3.2.2.

420.3.2.3 Stress in Bonded Prestressed Reinforcement at Nominal Flexural Strength, f_{ps}

420.3.2.3.1 As an alternative to a more accurate calculation of f_{ps} based on strain compatibility, values of f_{ps} calculated in accordance with Eq. 420.3.2.3.1 shall be

permitted for members with bonded prestressed reinforcement if all prestressed reinforcement is in the tension zone and $f_{ps} \geq 0.5f_{pu}$.

$$f_{ps} = f_{pu} \left\{ 1 - \frac{\gamma_p}{\beta_1} \left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p} \frac{f_y}{f'_c} (\rho - \rho') \right] \right\} \quad (420.3.2.3.1)$$

Where γ_p is in accordance with Table 420.3.2.3.1.

If compression reinforcement is considered for the calculation of f_{ps} by Eq. 420.3.2.3.1, (a) and (b) shall be satisfied.

- a. If d' exceeds $0.15d_p$, the compression reinforcement shall be neglected in Eq. 420.3.2.3.1.

- b. If compression reinforcement is included in Eq. 420.3.2.3.1, the term

$$\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p} \frac{f_y}{f'_c} (\rho - \rho') \right]$$

shall not be taken less than 0.17.

Table 420.3.2.3.1
Values of γ_p for Use in Equation (Sect. 420.3.2.3.1)

f_{py}/f_{pu}	γ_p
≥ 0.80	0.55
≥ 0.85	0.40
≥ 0.90	0.28

420.3.2.3.2 For pretensioned strands, the strand design stress at sections of members located within ℓ_d from the free end of strand shall not exceed that calculated in accordance with Section 425.4.8.3.

420.3.2.4 Stress in Unbonded Prestressed Reinforcement at Nominal Flexural Strength, f_{ps}

420.3.2.4.1 As an alternative to a more accurate calculation of f_{ps} , values of f_{ps} calculated in accordance with Table 420.3.2.4.1 shall be permitted for members prestressed with unbonded tendons if $f_{se} \geq 0.5f_{pu}$.

Table 420.3.2.4.1
Approximate Values of f_{ps} at Nominal Flexural Strength for Unbonded Tendons

ℓ_n/h	f_{ps}	
≤ 35	The least of:	$f_{se} + 70 + f'_c/(100\rho_p)$
		$f_{se} + 420$
		f_{py}
> 35	The least of	$f_{se} + 70 + f'_c/(300\rho_p)$
		$f_{se} + 210$
		f_{py}

420.3.2.5 Permissible Tensile Stresses in Prestressed Reinforcement

420.3.2.5.1 The tensile stress in prestressed reinforcement shall not exceed the limits in Table 420.3.2.5.1.

Table 420.3.2.5.1
Maximum Permissible Tensile Stresses in Prestressed Reinforcement

Stage	Location	Maximum tensile stress	
During stressing	At jacking end	Least of:	$0.94f_{py}$
			$0.80f_{pu}$
			Maximum jacking force recommended by the supplier of anchorage device
Immediately after force transfer	At post-tensioning anchorage devices and couplers	$0.70f_{pu}$	

420.3.2.6 Prestress Losses

420.3.2.6.1 Prestress losses shall be considered in the calculation of the effective tensile stress in the prestressed reinforcement, f_{se} , and shall include (a) through (f):

- Prestressed reinforcement seating at transfer;
- Elastic shortening of concrete;
- Creep of concrete;
- Shrinkage of concrete;
- Relaxation of prestressed reinforcement;
- Friction loss due to intended or unintended curvature in post-tensioning tendons.

420.3.2.6.2 Calculated friction loss in post-tensioning tendons shall be based on experimentally determined wobble and curvature friction coefficients.

420.3.2.6.3 Where loss of prestress in a member is anticipated due to connection of the member to adjoining construction, such loss of prestress shall be included in design calculations.

420.4 Structural Steel, Pipe, and Tubing for Composite Columns

420.4.1 Material Properties

420.4.1.1 Structural steel other than steel pipe or tubing used in composite columns shall conform to (a), (b), (c), (d), or (e):

- a. ASTM A36M – carbon steel;
- b. ASTM A242M – high-strength low-alloy steel;
- c. ASTM A572M – high-strength, low-alloy, columbium-vanadium steel;
- d. ASTM A588M – high-strength, low-alloy, 345 MPa steel;
- e. ASTM A992M – structural shapes.

420.4.1.2 Steel pipe or tubing used in composite columns to encase a concrete core shall conform to (a), (b), (c) or (d):

- a. ASTM A53M Grade B – black steel, hot-dipped, zinc-coated;
- b. ASTM A500M – cold-formed, welded, seamless;
- c. ASTM A501M – hot-formed, welded, seamless.
- d. ASTM A1085 – cold-formed, welded.

420.4.2 Design Properties

420.4.2.1 For structural steel in composite columns, maximum value of f_y shall be in accordance with the appropriate ASTM specifications in Section 420.4.1.

420.4.2.2 For structural steel used in composite columns with a structural steel core, value of f_y shall not exceed 345 MPa.

420.5 Headed Shear Stud Reinforcement

420.5.1 Headed shear stud reinforcement and stud assemblies shall conform to ASTM A1044M.

420.6 Provisions for Durability of Steel Reinforcement

420.6.1 Specified Concrete Cover

420.6.1.1 Unless the general building code requires a greater concrete cover for fire protection, the minimum specified concrete cover shall be in accordance with Section 420.6.1.2 through 420.6.1.4.

420.6.1.2 It shall be permitted to consider concrete floor finishes as part of required cover for non-structural purposes.

420.6.1.3 Specified Concrete Cover Requirements

420.6.1.3.1 Non-prestressed cast-in-place concrete members shall have specified concrete cover for reinforcement at least that given in Table 420.6.1.3.1.

Table 420.6.1.3.1
Specified Concrete Cover for Cast-in-Place Non-Prestressed Concrete Members

Concrete exposure	Member	Reinforcement	Specified cover, mm.
Cast against and permanently in contact with ground	All	All	75
Exposed to weather or in contact with ground	All	20 mm ϕ through 58 mm ϕ bars	50
		16 mm ϕ bar, MW200 or MD200 wire, and smaller	40
Not exposed to weather or in contact with ground	Slabs, joists, and walls	40 mm ϕ and 58 mm ϕ bars	40
		36 mm ϕ bar and smaller	20
	Beams, columns, pedestals, and tension ties	Primary reinforcement, stirrups, ties, spirals, and hoops	40

Table 420.6.1.3.2
Specified Concrete Cover for Cast-in-Place Prestressed Concrete Members

Concrete exposure	Member	Reinforcement	Specified cover, mm.
Cast against and permanently in contact with ground	All	All	75
Exposed to weather or in contact with ground	Slabs, joists, and walls	All	25
	All other	All	40
Not exposed to weather or in contact with ground	Slabs, joists, and walls	All	20
	Beams, columns, and tension ties	Primary reinforcement	40
		Stirrups, ties, spirals, and hoops	25

Table 420.6.1.3.3
Specified Concrete Cover for Precast Non-Prestressed
or Prestressed Concrete Members
Manufactured Under Plant Conditions

Concrete exposure	Member	Reinforcement	Specified cover, mm.
Exposed to weather or in contact with ground	Walls	40 mm ϕ and 58 mm ϕ bars; tendons larger than 40 mm diameter	40
		36 mm ϕ bars and smaller; MW200 and MD200 wire and smaller; tendons and strands 40 mm ϕ and smaller	20
	All other	40 mm ϕ and 58 mm ϕ bars; tendons larger than 40 mm ϕ	50
		20 mm ϕ through 36 mm ϕ bars; tendons and strands larger than 16 mm ϕ through 40 mm ϕ	40
		16 mm ϕ bar, MW200 or MD200 wire, and smaller; tendons and strands 16 mm ϕ and smaller	32
Not exposed to weather or in contact with ground	Slabs, joists, and walls	40 mm ϕ and 58 mm ϕ bars; tendons larger than 40 mm ϕ	45
		Tendons and strands 40 mm ϕ and smaller	20
		36 mm ϕ bar, MW200 or MD200 wire, and smaller	16
	Beams, columns, pedestals, and tension ties	Primary reinforcement	Greater of d_b and 16 mm and need not exceed 40 mm
		Stirrups, ties, spirals, and hoops	10

420.6.1.3.2 Cast-in-place prestressed concrete members shall have specified concrete cover for reinforcement, ducts and end fittings at least that given in Table 420.6.1.3.2.

420.6.1.3.3 Precast non-prestressed or prestressed concrete members manufactured under plant conditions shall have specified concrete cover for reinforcement, ducts, and end fittings at least that given in Table 420.6.1.3.3.

420.6.1.3.4 For bundled bars, specified concrete cover shall be at least the smaller of (a) and (b):

- The equivalent diameter of the bundle;
- 50 mm.

and for concrete cast against and permanently in contact with ground, the specified cover shall be 75 mm.

420.6.1.3.5 For headed shear stud reinforcement, specified concrete cover for the heads or base rails shall be at least that required for the reinforcement in the member.

420.6.1.4 Specified Concrete Cover Requirements for Corrosive Environments

420.6.1.4.1 In corrosive environments or other severe exposure conditions, the specified concrete cover shall be increased as deemed necessary. The applicable requirements for concrete based on exposure categories in Section 419.3 shall be satisfied, or other protection shall be provided.

420.6.1.4.2 For prestressed concrete members classified as Class T or C in Section 424.5.2 and exposed to corrosive environments or other severe exposure categories such as those given in Section 419.3, the specified concrete cover for prestressed reinforcement shall be at least 1.5 times the cover in Section 420.6.1.3.2 for cast-in-place members and in Section 420.6.1.3.3 for precast members.

420.6.1.4.3 If the pre-compressed tensile zone is not in tension under sustained loads, Section 420.6.1.4.2 need not be satisfied.

420.6.2 Non-Prestressed Coated Reinforcement

420.6.2.1 Non-prestressed coated reinforcement shall conform to Table 420.6.2.1.

Table 420.6.2.1
Non-Prestressed Coated Reinforcement

Type of Coating	Applicable ASTM specifications		
	Bar	Wire	Welded wire
Zinc-coated	A767M	Not permitted	A1060M
Epoxy-coated	A775M or A934M	A884M	A884M
Zinc and epoxy dual coated	A1055M	Not permitted	Not permitted

420.6.2.2 Deformed bars to be zinc-coated, epoxy-coated, or zinc and epoxy dual-coated shall conform to Sections 420.2.1.3 (a), (b) or (c).

420.6.2.3 Wire and welded wire reinforcement to be epoxy-coated shall conform to Section 420.2.1.7(a).

420.6.3 Corrosion Protection for Unbonded Prestressing Reinforcement

420.6.3.1 Unbonded prestressing reinforcement shall be encased in sheathing, and the space between the strand and the sheathing shall be completely filled with a material formulated to inhibit corrosion. Sheathing shall be watertight and continuous over the unbonded length.

420.6.3.2 In corrosive environments, the sheathing shall be connected to all stressing, intermediate, and fixed anchorages in a watertight fashion.

420.6.3.3 Unbonded single-strand tendons shall be protected against corrosion in accordance with ACI 423.7.

420.6.4 Corrosion Protection for Grouted Tendons

420.6.4.1 Ducts for grouted tendons shall be grout-tight and non-reactive with concrete, prestressing reinforcement, grout, and corrosion inhibitor admixtures.

420.6.4.2 Ducts shall be maintained free of water.

420.6.4.3 Ducts for grouted single-wire, single-strand, or single-bar tendons shall have an inside diameter at least 6mm larger than the diameter of the prestressing reinforcement.

420.6.4.4 Ducts for grouted multiple wire, multiple strand, or multiple bar tendons shall have an inside cross-sectional area at least two times the cross-sectional area of the prestressing reinforcement.

420.6.5 Corrosion Protection for Post-Tensioning Anchorages, Couplers, and End Fittings

420.6.5.1 Anchorages, couplers, and end fittings shall be protected to provide long-term resistance to corrosion.

420.6.6 Corrosion Protection for External Post-Tensioning

420.6.6.1 External tendons and tendon anchorage regions shall be protected against corrosion.

420.7 Embedments

420.7.1 Embedments shall not significantly impair the strength of the structure and shall not reduce fire protection.

420.7.2 Embedment materials shall not be harmful to concrete or reinforcement.

420.7.3 Aluminum embedments shall be coated or covered to prevent aluminum-concrete reaction and electrolytic action between aluminum and steel.

420.7.4 Reinforcement with an area at least 0.002 times the area of the concrete section shall be provided perpendicular to pipe embedments.

420.7.5 Specified concrete cover for pipe embedments with their fittings shall be at least 40 mm for concrete exposed to earth or weather, and at least 20mm for concrete not exposed to weather, or not in contact with ground.

420.7.6 Use of Quenched Tempered Thermo-Mechanically Treated (QT/TMT) Reinforcing Bars in Structures Located in Seismic Zone 4

420.7.6.1 (NZS 3101 5.3.2.1) Reinforcing bars to comply with AS/NZS 4671. Reinforcement shall be manufactured using either the micro-alloy process or the in-line quenched and tempered process. However, where the in-line quenched and tempered process, or equivalent is used, the restrictions of Section 420.7.6.2 shall apply.

420.7.6.2 (NZS 3101 5.3.2.2) Restrictions on In-line Quenched and Tempered Reinforcement:

Reinforcement bars manufactured by the in-line quenched and tempered process shall not be used when welding, galvanizing, hot bending, or threading of bars occurs.

420.7.6.3 (NZS 3101 C5.3.2)

It is important to note that any process involving heat e.g. welding, galvanizing, hot bending can adversely affect the mechanical properties of quenched and tempered reinforcing bars by modification of the micro-structure.

Threading of quenched and tempered bar removes some to all hardened outer layer resulting in a disproportionate loss of strength.

420.7.6.4 (NZS 3109 Amendment 2)

Quenched and tempered reinforcing bars shall not be straightened or rebent.

420.7.6.5 (AS/NZS 1554.3:2002)

Quenched and tempered reinforcing bars cannot be welded without strength loss. It is recommended that a suitable warning be added to the standard to this effect. This is covered in the amendment to NZS 3101.

420.7.7 Restrictions in the Use of Quenched Tempered Thermo-mechanically Treated (QT/TMT) Reinforcing Bars in Structures Located in Seismic Zone 4

420.7.7.1 The restrictions prohibit the use of quenched tempered (QT) and thermo-mechanically treated (TMT) reinforcing bars specifically where:

1. There is preheating greater than 275°C;
2. Bending of reinforcing bars requiring preheating;
3. Splicing of reinforcing bars that will require welding or lap or butt welded joints;
4. Threading of reinforcing bar ends for use of mechanical couplers;
5. Tack welding for grounding wires will be required.

420.7.7.2 Use of QT/TMT Reinforcing Bars Not Complying to the Foregoing Restrictions:

420.7.7.2.1 Design Engineers are reminded that the performance of QT/TMT rebars under cyclic loading could result in premature spalling and failure of the concrete and the premature failure of the QT/TMT reinforcing bars.

420.7.7.2.2 Civil Engineer-in-Charge of Construction shall inform the Design Engineer in writing if QT/TMT reinforcing bars are to be substituted to MA reinforcing bars.

**SECTION 421
STRENGTH REDUCTION
FACTORS****421.1 Scope**

421.1.1 This section shall apply to the selection of strength reduction factors used in design, except as permitted by Section 427.

421.2 Strength Reduction Factors for Structural Concrete Members and Connections

421.2.1 Strength reduction factors, ϕ , shall be in accordance with Table 421.2.1, except as modified by Sections 421.2.2, 421.2.3, and 421.2.4.

421.2.2 Strength reduction factor for moment, axial force, or combined moment and axial force shall be in accordance with Table 421.2.2.

Table 421.2.1
Strength Reduction Factors, ϕ

Action or Structural Element		ϕ	Exceptions
(a)	Moment, axial force, or combined moment and axial force	0.65 to 0.90 in accordance with Section 421.2.2	Near ends of pretensioned members where strands are not fully developed, ϕ shall be in accordance with Section 421.2.3.
(b)	Shear	0.75	Additional requirements are given in Section 421.2.4 for structures designed to resist earthquake effects.
(c)	Torsion	0.75	-
(d)	Bearing	0.65	-
(e)	Post-tensioned anchorage zones	0.85	-
(f)	Brackets and corbels	0.75	-
(g)	Struts, ties, nodal zones, and bearing areas designed in accordance with strut-and-tie method in Section 423	0.75	-
(h)	Components of connections of precast members controlled by yielding of steel elements in tension	0.90	-
(i)	Plain concrete elements	0.60	-
(j)	Anchors in concrete elements	0.45 to 0.75 in accordance with Section 417	-

Table 421.2.2
Strength Reduction Factor, ϕ , for Moment, Axial Force, or Combined Moment and Axial Force

Net tensile stain, ϵ_t	Classification	ϕ			
		Type of transverse reinforcement			
		Spirals conforming to Sect. 425.7.3		Other	
$\epsilon_t \leq \epsilon_{ty}$	Compression controlled	0.75	(a)	0.65	(b)
$\epsilon_{ty} < \epsilon_t < 0.005$	Transition ^[1]	$0.75 + 0.15 \frac{(\epsilon_t - \epsilon_{ty})}{(0.005 - \epsilon_{ty})}$	(c)	$0.65 + 0.25 \frac{(\epsilon_t - \epsilon_{ty})}{(0.005 - \epsilon_{ty})}$	(d)
$\epsilon_t \geq 0.005$	Tension controlled	0.90	(e)	0.90	(f)

^[1] For sections classified as transition, it shall be permitted to use ϕ corresponding to compression-controlled sections.

421.2.2.1 For deformed reinforcement, ϵ_{ty} shall be f_y/E_s . For Grade 280 deformed reinforcement, it shall be permitted to take ϵ_{ty} equal to 0.002.

$$\ell_{tr} = \left(\frac{f_{se}}{21} \right) d_b \quad (421.2.3)$$

421.2.2.2 For all prestressed reinforcement, ϵ_{ty} shall be taken as 0.002.

421.2.3 For sections in pretensioned members where strand is not fully developed, ϕ shall be calculated at each section in accordance with Table 421.2.3, where ℓ_{tr} is calculated using Eq. 421.2.3, ℓ_{db} is the debonded length at the end of the member, f_{se} is the effective stress in the prestressed reinforcement after allowance for all losses, and d_b is given in Section 425.4.8.1.

421.2.4 For structures that rely on elements in (a), (b), or (c) to resist earthquake effects, E , the value of ϕ for shear shall be modified in accordance with Sections 421.2.4.1 through 421.2.4.3

- Special moment frames;
- Special structural walls;
- Intermediate precast structural walls in structures assigned to seismic zone 4.

421.2.4.1 For any member designed to resist E , ϕ for shear shall be 0.60 if the nominal shear strength of the member is less than the shear corresponding to the development of the nominal moment strength of the member. The nominal moment strength shall be calculated considering the most critical factored axial loads and including E .

421.2.4.2 For diaphragms, ϕ for shear shall not exceed the least value of ϕ for shear used for the vertical components of the primary seismic-force-resisting system.

421.2.4.3 For beam-column joints and diagonally reinforced coupling beams, ϕ for shear shall be 0.85.

Table 421.2.3 Strength Reduction Factor, ϕ , for Section of Pretensioned Members

Condition near end of member	Stress in concrete under service load ^[1]	Distance from end of member to section under consideration	ϕ	
All strands bonded	Not applicable	$\leq \ell_{tr}$	0.75	(a)
		ℓ_{tr} to ℓ_d	Linear interpolation from 0.75 to 0.90 ^[2]	(b)
One or more Strands debonded	No tension calculated	$\leq (\ell_{db} + \ell_{tr})$	0.75	(c)
		$(\ell_{db} + \ell_{tr})$ to $(\ell_{db} + \ell_d)$	Linear interpolation from 0.75 to 0.90 ^[2]	(d)
	Tension calculated	$\leq (\ell_{db} + \ell_{tr})$	0.75	(e)
		$(\ell_{db} + \ell_{tr})$ to $(\ell_{db} + 2\ell_d)$	Linear interpolation from 0.75 to 0.90 ^[2]	(f)

^[1] Stress calculated using gross cross-sectional properties in extreme concrete fiber of pre-compressed tensile zone under service loads after allowance for all prestress losses at section under consideration.

^[2] It shall be permitted to use a strength reduction factor of 0.75.

SECTION 422

SECTIONAL STRENGTH

422.1 Scope

422.1.1 This section shall apply to calculating nominal strength at sections of members, including (a) through (g):

- a. Flexural strength;
- b. Axial strength or combined flexural and axial strength;
- c. One-way shear strength;
- d. Two-way shear strength;
- e. Torsional strength;
- f. Bearing;
- g. Shear friction.

422.1.2 Sectional strength requirements of this chapter shall be satisfied unless the member or region of the member is designed in accordance with Section 423.

422.1.3 Design strength at a section shall be taken as the nominal strength multiplied by the applicable strength reduction factor, ϕ , given in Section 421.

422.2 Design Assumptions for Moment and Axial Strength

422.2.1 Equilibrium and Strain Compatibility

422.2.1.1 Equilibrium shall be satisfied at each section.

422.2.1.2 Strain in concrete and non-prestressed reinforcement shall be assumed proportional to the distance from neutral axis.

422.2.1.3 Strain in prestressed concrete and in bonded and unbonded prestressed reinforcement shall include the strain due to effective prestress.

422.2.1.4 Changes in strain for bonded prestressed reinforcement shall be assumed proportional to the distance from neutral axis.

422.2.2 Design Assumptions for Concrete

422.2.2.1 Maximum strain at the extreme concrete compression fiber shall be assumed equal to **0.003**.

422.2.2.2 Tensile strength of concrete shall be neglected in flexural and axial strength calculations.

422.2.2.3 The relationship between concrete compressive stress and strain shall be represented by a rectangular, trapezoidal, parabolic, or other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

422.2.2.4 The equivalent rectangular concrete stress distribution in accordance with Sections 422.2.2.4.1 through 422.2.2.4.3 satisfies Section 422.2.2.3.

422.2.2.4.1 Concrete stress of $0.85f'_c$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a line parallel to the neutral axis located a distance α from the fiber of maximum compressive strain, as calculated by:

$$\alpha = \beta_1 c \quad (422.2.2.4.1)$$

422.2.2.4.2 Distance from the fiber of maximum compressive strain to the neutral axis, c , shall be measured perpendicular to the neutral axis.

422.2.2.4.3 Values of β_1 shall be in accordance with Table 422.2.2.4.3

Table 422.2.2.4.3
Values of β_1 for Equivalent Rectangular Concrete Stress Distribution

f'_c , MPa	β_1	
$17 \leq f'_c \leq 28$	0.85	(a)
$28 < f'_c < 55$	$0.85 \frac{0.05(f'_c - 28)}{7}$	(b)
$f'_c \geq 55$	0.65	(c)

422.2.3 Design Assumptions for Non-Prestressed Reinforcement

422.2.3.1 Deformed reinforcement used to resist tensile or compressive forces shall conform to Section 420.2.1.

422.2.3.2 Stress-strain relationship and modulus of elasticity for deformed reinforcement shall be idealized in accordance with Sections 420.2.2.1 and 420.2.2.2.

422.2.4 Design Assumptions for Prestressing Reinforcement

422.2.4.1 For members with bonded prestressing reinforcement conforming to Section 420.3.1, stress at

nominal flexural strength, f_{ps} , shall be calculated in accordance with Section 420.3.2.3.

422.2.4.2 For members with unbonded prestressing reinforcement conforming to Section 420.3.1, f_{ps} shall be calculated in accordance with Section 420.3.2.4.

422.2.4.3 If the embedded length of the prestressing strand is less than ℓ_d , the design strand stress shall not exceed the value given in Section 425.4.8.3, as modified by Section 425.4.8.1(b).

422.3 Flexural Strength

422.3.1 General

422.3.1.1 Nominal flexural strength M_n shall be calculated in accordance with the assumptions of Section 422.2.

422.3.2 Prestressed Concrete Members

422.3.2.1 Deformed reinforcement conforming to Section 420.2.1, provided in conjunction with prestressed reinforcement, shall be permitted to be considered to contribute to the tensile force and be included in flexural strength calculations at a stress equal to f_y .

422.3.2.2 Other non-prestressed reinforcement shall be permitted to be considered to contribute to the flexural strength if a strain compatibility analysis is performed to calculate stresses in such reinforcement.

422.3.3 Composite Concrete Members

422.3.3.1 Provisions of Section 422.3.3 apply to members constructed in separate placements but connected so that all elements resist loads as a unit.

422.3.3.2 For calculation of M_n for composite slabs and beams, use of the entire composite section shall be permitted.

422.3.3.3 For calculation of M_n for composite slabs and beams, no distinction shall be made between shored and unshored members.

422.3.3.4 For calculation of M_n for composite members where the specified concrete compressive strength of different elements varies, properties of the individual elements shall be used in design. Alternatively, it shall be permitted to use the value of f'_c for the element that results in the most critical value of M_n .

422.4 Axial Strength or Combined Flexural and Axial Strength

422.4.1 General

422.4.1.1 Nominal flexural and axial strength shall be calculated in accordance with the assumptions of Section 422.2.

422.4.2 Maximum Axial Compressive Strength

422.4.2.1 Nominal axial compressive strength, P_n , shall not exceed $P_{n,max}$, in accordance with Table 422.4.2.1, where P_o is calculated by Eq. 422.4.2.2 for non-prestressed members and composite steel and concrete members, and by Eq. 422.4.2.3 for prestressed members.

Table 422.4.2.1
Maximum Axial Strength

Member	Transverse Reinforcement	$P_{n,max}$	
Non-prestressed	Ties conforming to Section 422.4.2.4	$0.80 P_o$	(a)
	Spirals conforming to Section 422.4.2.5	$0.85 P_o$	(b)
Prestressed	Ties	$0.80 P_o$	(c)
	Spirals	$0.85 P_o$	(d)
Composite steel and concrete columns in accordance with Section 410	All	$0.85 P_o$	(e)

422.4.2.2 For non-prestressed members and composite steel and concrete members, P_o shall be calculated by

$$P_o = 0.85f'_c(A_g - A_{st}) + f_y A_{st} \quad (422.4.2.2)$$

where A_{st} is the total area of non-prestressed longitudinal reinforcement.

422.4.2.3 For prestressed members, P_o shall be calculated by:

$$P_o = 0.85f'_c(A_g - A_{st} - A_{pd}) + f_y A_{st} - (f_{se} - 0.003E_p)A_{pt} \quad (422.4.2.3)$$

Where A_{pt} is the total area of prestressing reinforcement, A_{pd} is the total area occupied by duct, sheathing, and prestressing reinforcement, and the value of f_{se} shall be at least $0.003E_p$. For grouted, post-tensioned tendons, it shall be permitted to assume A_{pd} equals A_{pt} .

422.4.2.4 Tie reinforcement for lateral support of longitudinal reinforcement in compression members shall satisfy Sections 410.7.6.2 and 425.7.2.

422.4.2.5 Spiral reinforcement for lateral support of longitudinal reinforcement in compression members shall satisfy Sections 410.7.6.3 and 425.7.3.

422.4.3 Maximum Axial Tensile Strength

422.4.3.1 Nominal axial tensile strength of a non-prestressed, composite, or prestressed member, P_{nt} , shall not be taken greater than $P_{nt,max}$, calculated by:

$$P_{nt,max} = f_y A_{st} + (f_{se} + \Delta f_p) A_{pt} \quad (422.4.3.1)$$

where $(f_{se} + \Delta f_p)$ shall not exceed f_{py} , and A_{pt} is zero for non-prestressed members.

422.5 One-way Shear Strength

422.5.1 General

422.5.1.1 Nominal one-way shear strength at a section, V_n , shall be calculated by:

$$V_n = V_c + V_s \quad (422.5.1.1)$$

422.5.1.2 Cross-sectional dimensions shall be selected to satisfy Eq. 422.5.1.2.

$$V_u \leq \phi(V_c + 0.67\sqrt{f'_c}b_wd) \quad (422.5.1.2)$$

422.5.1.3 For non-prestressed members, V_c shall be calculated in accordance with Sections 422.5.5, 422.5.6, or 422.5.7.

422.5.1.4 For prestressed members, V_c , V_{ci} , and V_{cw} shall be calculated in accordance with Section 422.5.8 or 422.5.9.

422.5.1.5 For calculation of V_c , V_{ci} , and V_{cw} , λ shall be in accordance with Section 419.2.4.

422.5.1.6 V_s shall be calculated in accordance with Section 422.5.10.

422.5.1.7 Effect of any openings in members shall be considered in calculating V_n .

422.5.1.8 Effect of axial tension due to creep and shrinkage in restrained members shall be considered in calculating V_c .

422.5.1.9 Effect of inclined flexural compression in variable depth members shall be permitted to be considered in calculating V_c .

422.5.2 Geometric Assumptions

422.5.2.1 For calculation of V_c and V_s in prestressed members, d shall be taken as the distance from the extreme compression fiber to the centroid of prestressed and any non-prestressed longitudinal reinforcement but need not be taken less than $0.8h$.

422.5.2.2 For calculation of V_c and V_s in solid, circular sections, d shall be permitted to be taken as 0.8 times the diameter and b_w shall be permitted to be taken as the diameter.

422.5.3 Limiting Material Strengths

422.5.3.1 The value of $\sqrt{f'_c}$ used to calculate V_c , V_{ci} , and V_{cw} for one-way shear shall not exceed 8.3MPa, unless allowed in Section 422.5.3.2.

422.5.3.2 Values of $\sqrt{f'_c}$ greater than 8.3MPa shall be permitted in calculating V_c , V_{ci} , and V_{cw} for reinforced or prestressed concrete beams and concrete joist construction having minimum web reinforcement in accordance with Section 409.6.3.3 or 409.6.4.2.

422.5.3.3 The values of f_y and f_{yt} used to calculate V_s shall not exceed the limits in Section 420.2.2.4.

422.5.4 Composite Concrete Members

422.5.4.1 This section shall apply to members constructed in separate placements but connected so that all elements resist loads as a unit.

422.5.4.2 For calculation of V_n for composite members, no distinction shall be made between shored and unshored members.

422.5.4.3 For calculation of V_n for composite members where the specified concrete compressive strength, unit weight, or other properties of different elements vary, properties of the individual elements shall be used in design. Alternatively, it shall be permitted to use the properties of the element that results in the most critical value of V_n .

422.5.4.4 If an entire composite member is assumed to resist vertical shear, it shall be permitted to calculate V_c assuming a monolithically cast member of the same cross-sectional shape.

422.5.4.5 If an entire composite member is assumed to resist vertical shear, it shall be permitted to calculate V_s assuming a monolithically cast member of the same cross-sectional shape if shear reinforcement is fully anchored into the interconnected elements in accordance with Section 425.7.

422.5.5 V_c for Non-Prestressed Members without Axial Force

422.5.5.1 For non-prestressed members without axial force, V_c shall be calculated by:

$$V_c = 0.17\lambda\sqrt{f'_c}b_wd \quad (422.5.5.1)$$

unless a more detailed calculation is made in accordance with Table 422.5.5.1.

Table 422.5.5.1
Detailed Method for Calculating V_c

V_c		
Least of (a), (b), and (c):	$\left(0.16\lambda\sqrt{f'_c} + 17\rho_w \frac{V_u d}{M_u}\right) b_w d$ ^[1]	(a)
	$\left(0.16\lambda\sqrt{f'_c} + 17\rho_w\right) b_w d$	(b)
	$0.29\lambda\sqrt{f'_c}b_wd$	(c)

^[1] M_u occurs simultaneously with V_u at the section considered

422.5.6 V_c for Non-Prestressed Members with Axial Compression

422.5.6.1 For non-prestressed members with axial compression, V_c shall be calculated by:

$$V_c = 0.17\left(1 + \frac{N_u}{14A_g}\right)\lambda\sqrt{f'_c}b_wd \quad (422.5.6.1)$$

unless a more detailed calculation is made in accordance with Table 422.5.6.1, where N_u is positive for compression.

Table 422.5.6.1
Detailed Method for Calculating V_c for Non-Prestressed Members with Axial Compression

V_c		
Lesser of (a), (b), and (c):	$\left(0.16\lambda\sqrt{f'_c} + 17\rho_w \frac{V_u d}{M_u - N_u \frac{4h-d}{8}}\right) b_w d$ ^[1]	(a)
	Equation not applicable if $M_u - N_u \left(\frac{4h-d}{8}\right) \leq 0$	(b)
	$0.29\lambda\sqrt{f'_c}b_wd \sqrt{1 + \frac{0.29N_u}{A_g}}$	

^[1] M_u occurs simultaneously with V_u at the section considered

422.5.7 V_c for Non-Prestressed Members with Significant Axial Tension

422.5.7.1 For non-prestressed members with significant axial tension, V_c shall be calculated by:

$$V_c = 0.17\left(1 + \frac{N_u}{3.5A_g}\right)\lambda\sqrt{f'_c}b_wd \quad (422.5.7.1)$$

where N_u is negative for tension, and V_c shall not be less than zero.

422.5.8 V_c for Prestressed Members

422.5.8.1 This section shall apply to the calculation of V_c for post-tensioned and pretensioned members in regions where the effective force in the prestressed reinforcement is fully transferred to the concrete. For regions of pretensioned members where the effective force in the prestressed reinforcement is not fully transferred to the concrete, Section 422.5.9 shall govern the calculation of V_c .

422.5.8.2 For prestressed flexural members with $A_{ps}f_{se} \geq 0.4(A_{ps}f_{pu} + A_s f_y)$, V_c shall be calculated in accordance with Table 422.5.8.2, but need not be less than the value calculated by Eq. 422.5.5.1. Alternatively, it shall be permitted to calculate V_c in accordance with Section 422.5.8.3.

Table 422.5.8.2
Approximate Method for Calculating V_c

V_c		
Least of (a), (b), and (c):	$\left(0.05\lambda\sqrt{f'_c} + 4.8 \frac{V_u d_p}{M_u}\right) b_w d$ ^[1]	(a)
	$\left(0.05\lambda\sqrt{f'_c} + 4.8\right) b_w d$	(b)
	$0.42\lambda\sqrt{f'_c}b_wd$	(c)

^[1] M_u occurs simultaneously with V_u at the section considered

422.5.8.3 For prestressed members, V_c shall be permitted to be the lesser of V_{ci} calculated in accordance with Section 422.5.8.3.1 and V_{cw} calculated in accordance with Section 422.5.8.3.2 or 422.5.8.3.3.

422.5.8.3.1 The flexure-shear strength, V_{ci} , shall be the greater of (a) and (b):

$$a. \quad V_{ci} = 0.05\lambda\sqrt{f'_c}b_wd + V_d + \frac{V_iM_{cre}}{M_{max}} \quad (422.5.8.3.1a)$$

$$b. \quad V_{ci} = 0.14\lambda\sqrt{f'_c}b_wd \quad (422.5.8.3.1b)$$

where d_p need not be taken less than $0.80h$, the values of M_{max} and V_i shall be calculated from the load combinations causing maximum factored moment to occur at section considered, and M_{cre} shall be calculated by:

$$M_{cre} = \left(\frac{1}{y_t}\right)(0.5\lambda\sqrt{f'_c} + f_{pe} - f_d) \quad (422.5.8.3.1c)$$

422.5.8.3.2 The web-shear strength, V_{cw} , shall be calculated by:

$$V_{cw} = (0.29\lambda\sqrt{f'_c} + 0.3f_{pc})b_wd_p + V_p \quad (422.5.8.3.2)$$

where d_p need not be taken less than $0.80h$ and V_p is the vertical component of the effective prestress.

422.5.8.3.3 As an alternative to Section 422.5.8.3.2, it shall be permitted to calculate V_{cw} as the shear force corresponding to dead load plus live load that results in a principal tensile stress of $0.33\lambda\sqrt{f'_c}$ at location (a) or (b):

- Where the centroidal axis of the prestressed cross section is in the web, the principal tensile stress shall be calculated at the centroidal axis;
- Where the centroidal axis of the prestressed cross section is in the flange, the principal tensile stress shall be calculated at the intersection of the flange and the web.

422.5.8.3.4 In composite members, the principal tensile stress in Section 422.5.8.3.3 shall be calculated using the cross section that resists live load.

422.5.9 V_c for Pretensioned Members in Regions of Reduced Prestress Force

422.5.9.1 When calculating V_c , the transfer length of prestressed reinforcement ℓ_{tr} , shall be assumed to be $50d_b$ for strand and $100d_b$ for wire.

422.5.9.2 If bonding of strands extends to the end of the member, the effective prestress force shall be assumed to vary linearly from zero at the end of the prestressed reinforcement to a maximum at a distance ℓ_{tr} from the end of the prestressed reinforcement.

422.5.9.3 At locations corresponding to a reduced effective prestress force in Section 422.5.9.2, V_c shall be calculated in accordance with (a) through (c):

- The reduced effective prestress force shall be used to determine the applicability of Section 422.5.8.2;
- The reduced effective prestress force shall be used to calculate V_{cw} in Section 422.5.8.3;
- The value of V_c calculated using Section 422.5.8.2 shall not exceed the value of V_{cw} calculated using the reduced effective prestress force.

422.5.9.4 If bonding of strands does not extend to the end of the member, the effective prestress force shall be assumed to vary linearly from zero at the point where bonding commences to a maximum at a distance ℓ_{tr} from the point.

422.5.9.5 At locations corresponding to a reduced effective prestress force according to Section 422.5.9.4, V_c shall be calculated in accordance with (a) through (c):

- The reduced effective prestress force shall be used to determine the applicability of Section 422.5.8.2;
- The reduced effective prestress force shall be used to calculate V_c in accordance with Section 422.5.8.3;
- The value of V_c calculated using Section 422.5.8.2 shall not exceed the value of V_{cw} calculated using the reduced effective prestress force.

422.5.10 One-way Shear Reinforcement

422.5.10.1 At each section where $V_u > \phi V_c$, transverse reinforcement shall be provided such that Eq. 422.5.10.1 is satisfied.

$$V_s \geq \frac{V_u}{\phi} - V_c \quad (422.5.10.1)$$

422.5.10.2 For one-way members reinforced with transverse reinforcement, V_s shall be calculated in accordance with Section 422.5.10.5.

422.5.10.3 For one-way members reinforced with bent-up longitudinal bars, V_s shall be calculated in accordance with Section 422.5.10.6.

422.5.10.4 If more than one type of shear reinforcement is provided to reinforce the same portion of a member, V_s shall be the sum of the V_s values for the various types of shear reinforcement.

422.5.10.5 One-way Shear Strength Provided by Transverse Reinforcement

422.5.10.5.1 In non-prestressed and prestressed members, shear reinforcement satisfying (a), (b), or (c) shall be permitted:

- Stirrups, ties, or hoops perpendicular to longitudinal axis of member;
- Axial welded wire reinforcement with wires located perpendicular to longitudinal axis of member;
- Spiral reinforcement.

422.5.10.5.2 Inclined stirrups making an angle of at least 45 degrees with the longitudinal axis of the member and crossing the plane of the potential shear crack shall be permitted to be used as shear reinforcement in non-prestressed members.

422.5.10.5.3 V_s for shear reinforcement in Section 422.5.10.5.1 shall be calculated by:

$$V_s = \frac{A_v f_{yt} d}{s} \quad (422.5.10.3)$$

where s is the spiral pitch or the longitudinal spacing of the shear reinforcement and A_v is given in Section 422.5.10.5.5 or 422.5.10.5.6.

422.5.10.5.4 V_s for shear reinforcement in Section 422.5.10.5.2 shall be calculated by:

$$V_s = \frac{A_v f_{yt} (\sin \alpha + \cos \alpha) d}{s} \quad (422.5.10.4)$$

where α is the angle between the inclined stirrups and the longitudinal axis of the member, s is measured parallel to the longitudinal reinforcement, and A_v is given in Section 422.5.10.5.5.

422.5.10.5.5 For each rectangular tie, stirrup, hoop, or crosstie, A_v shall be the effective area of all bar legs or wires within spacing s .

422.5.10.5.6 For each circular tie or spiral, A_v shall be two times the area of the bar or wire within spacing s .

422.5.10.6 One-Way Shear Strength Provided by Bent-Up Longitudinal Bars

422.5.10.6.1 The center three-fourths of the inclined portion of bent-up longitudinal bars shall be permitted to be used as shear reinforcement in non-prestressed members if the angle α between the bent-up bars and the longitudinal axis of the member is at least 30 degrees.

422.5.10.6.2 If shear reinforcement consists of a single bar or a single group of parallel bars having an area A_v , all bent the same distance from the support, V_s shall be the lesser of (a) and (b):

$$a. \quad V_s = A_v f_y \sin \alpha \quad (422.5.10.6.2a)$$

$$b. \quad V_s = 0.25 \sqrt{f'_c} b_w d \quad (422.5.10.6.2b)$$

where α is the angle between bent-up reinforcement and longitudinal axis of the member.

422.5.10.6.3 If shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, V_s shall be calculated by Eq. 422.5.10.5.4.

422.6 Two-Way Shear Strength

422.6.1 General

422.6.1.1 Provisions Sections 422.6.1 through 422.6.8 apply to the nominal shear strength of two-way members with and without shear reinforcement. Where structural steel I- or channel-shaped sections are used as shear heads, two-way members shall be designed for shear in accordance with Section 422.6.9.

422.6.1.2 Nominal shear strength for two-way members without shear reinforcement shall be calculated by

$$v_n = v_c \quad (422.6.1.2)$$

422.6.1.3 Nominal shear strength for two-way members with shear reinforcement other than shear heads shall be calculated by

$$v_n = v_c + v_s \quad (422.6.1.3)$$

422.6.1.4 Two-way shear shall be resisted by a section with a depth d and an assumed critical perimeter b_o as defined in Section 422.6.4.

422.6.1.5 v_c for two-way shear shall be calculated in accordance with Section 422.6.5. For two-way members with shear reinforcement, v_c shall not exceed the limits in Section 422.6.6.1.

422.6.1.6 For calculation of v_c , λ shall be in accordance with Section 419.2.4.

422.6.1.7 For two-way members reinforced with single- or multiple-leg stirrups, v_s shall be calculated in accordance with Section 422.6.7.

422.6.1.8 For two-way members reinforced with headed shear stud reinforcement, v_s shall be calculated in accordance with Section 422.6.8.

422.6.2 Effective Depth

422.6.2.1 For calculation of v_c and v_s for two-way shear, d shall be the average of the effective depths in the two orthogonal directions.

422.6.2.2 For prestressed, two-way members, d need not be taken less than $0.8h$.

422.6.3 Limiting Material Strengths

422.6.3.1 The value of $\sqrt{f'_c}$ used to calculate v_c for two-way shear shall not exceed 8.3MPa.

422.6.3.2 The value of f_{yt} used to calculate v_s shall not exceed the limits in Section 420.2.2.4.

422.6.4 Critical Sections for Two-way Members

422.6.4.1 For two-way shear, critical sections shall be located so that the perimeter b_o is a minimum but need not be closer than $d/2$ to (a) and (b):

- Edges or corners of columns, concentrated loads, or reaction areas;
- Changes in slab or footing thickness, such as edges of capitals, drop panels, or shear caps.

422.6.4.1.1 For square or rectangular columns, concentrated loads, or reaction areas, critical sections for two-way shear in accordance with Section 422.6.4.1(a)

and (b) shall be permitted to be defined assuming straight sides.

422.6.4.1.2 For a circular or regular polygon-shaped column, critical sections for two-way shear in accordance with Section 422.6.4.1 (a) and (b) shall be permitted to be defined assuming a square column of equivalent area.

422.6.4.2 For two-way members reinforced with headed shear reinforcement or single- or multi-leg stirrups, a critical section with perimeter b_o located $d/2$ beyond the outermost peripheral line of shear reinforcement shall also be considered. The shape of this critical section shall be a polygon selected to minimize b_o .

422.6.4.3 If an opening is located within a column strip or closer than $10h$ from a concentrated load or reaction area, a portion of b_o enclosed by straight lines projecting from the centroid of the column, concentrated load or reaction area and tangent to the boundaries of the opening shall be considered ineffective.

422.6.5 Two-way Shear Strength Provided by Concrete

422.6.5.1 For non-prestressed two-way members, v_c shall be calculated in accordance with Section 422.6.5.2. For prestressed two-way members, v_c shall be calculated in accordance with (a) or (b):

- Section 422.6.5.2
- Section 422.6.5.5, if the conditions of Section 422.6.5.4 are satisfied

422.6.5.2 v_c shall be calculated in accordance with Table 422.6.5.2.

Table 422.6.5.2
Calculation of v_c for Two-way Shear

v_c		
Least of (a), (b), and (c):	$0.33\lambda \sqrt{f'_c}$	(a)
	$0.17 \left(1 + \frac{2}{\beta}\right) \lambda \sqrt{f'_c}$	(b)
	$0.083 \left(2 + \frac{\alpha_s d}{b_o}\right) \lambda \sqrt{f'_c}$	(c)

Note: β is the ratio of long side to short side of the column, concentrated load, or reaction area and α_s is given in Section 422.6.5.3.

422.6.5.3 The value of α_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns.

422.6.5.4 For prestressed, two-way members, it shall be permitted to calculate v_c using Section 422.6.5.5 provided that (a) through (c) are satisfied:

- Bonded reinforcement is provided in accordance with Sections 408.6.2.3 and 408.7.5.3;
- No portion of the column cross section is closer to a discontinuous edge than four times the slab thickness h ;
- Effective prestress, f_{pc} , in each direction is not less than 0.9 MPa.

422.6.5.5 For prestressed, two-way members conforming to Section 422.6.5.4, v_c shall be permitted to be the lesser of (a) and (b):

- $$v_c = 0.29\lambda \sqrt{f'_c} + 0.3f_{pc} + \frac{V_p}{b_o d} \quad (422.6.5.5a)$$
- $$v_c = 0.083\lambda \sqrt{f'_c} \left(1.5 + \frac{\alpha_s d}{b_o}\right) + 0.3f_{pc} + \frac{V_p}{b_o d} \quad (422.6.5.5b)$$

where α_s is given in Section 422.6.5.3, the value of f_{pc} is the average of f_{pc} in the two directions and shall not exceed 3.5 MPa, V_p is the vertical component of all effective prestress forces crossing the critical section, and the value of $\sqrt{f'_c}$ shall not exceed 5.8 MPa.

422.6.6 Maximum Shear for Two-way Members with Shear Reinforcement

422.6.6.1 For two-way members with shear reinforcement, the value of v_c calculated at critical sections shall not exceed the limits in Table 422.6.6.1.

Table 422.6.6.1
Maximum v_c for Two-Way Members with Shear Reinforcement

Type of shear reinforcement	Maximum v_c at critical sections defined in Section 422.6.4.1		Maximum v_c at critical section defined in Section 422.6.4.2	
Stirrups	$0.17\lambda \sqrt{f'_c}$	(a)	$0.17\lambda \sqrt{f'_c}$	(b)
Headed shear stud reinforcement	$0.25\lambda \sqrt{f'_c}$	(c)	$0.17\lambda \sqrt{f'_c}$	(d)

422.6.6.2 For two-way members with shear reinforcement, effective depth shall be selected such that v_u calculated at critical sections does not exceed the values in Table 422.6.6.2

Table 422.6.6.2
Maximum v_u for Two-Way Members with Shear Reinforcement

Type of shear reinforcement	Maximum v_u at critical sections defined in Section 422.6.4.1	
Stirrups	$\phi 0.50 \sqrt{f'_c}$	(a)
Headed shear stud reinforcement	$\phi 0.66 \sqrt{f'_c}$	(b)

422.6.7 Two-way Shear Strength Provided by Single- or Multiple-leg Stirrups

422.6.7.1 Single- or multiple-leg stirrups fabricated from bars or wires shall be permitted to be used as shear reinforcement in slabs and footings satisfying (a) and (b):

- d is at least 150 mm;
- d is at least $16d_b$, where d_b is the diameter of the stirrups.

422.6.7.2 For two-way members with stirrups, v_s shall be calculated by:

$$v_s = \frac{A_v f_{yt}}{b_o s} \quad (422.6.7.2)$$

where A_v is the sum of the area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the column section, and s is the spacing of the peripheral lines of shear reinforcement in the direction perpendicular to the column face.

422.6.8 Two-way Shear Strength Provided by Headed Shear Stud Reinforcement

422.6.8.1 Headed shear stud reinforcement shall be permitted to be used as shear reinforcement in slabs and footings if the placement and geometry of the headed shear stud reinforcement satisfies Section 408.7.7.

422.6.8.2 For two-way members with headed shear stud reinforcement, v_s shall be calculated by:

$$v_s = \frac{A_v f_{yt}}{b_o s} \quad (422.6.8.2)$$

where A_v is the sum of the area of all shear studs on one peripheral line that is geometrically similar to the perimeter of the column section, and s is the spacing of the peripheral lines of headed shear stud reinforcement in the direction perpendicular to the column face.

422.6.8.3 If headed shear stud reinforcement is provided, A_v/s shall satisfy:

$$\frac{A_v}{s} \geq 0.17 \sqrt{f'_c} \frac{b_o}{f_{yt}} \quad (422.6.8.3)$$

422.6.9 Design Provisions for Two-way Members with Shearheads.

422.6.9.1 Each shearhead shall consist of steel shapes fabricated with a full penetration weld into identical arms at right angles. Shearhead arms shall not be interrupted within the column section.

422.6.9.2 A shearhead shall not be deeper than 70 times the web thickness of the steel shape.

422.6.9.3 The ends of each shearhead arm shall be permitted to be cut at angles of at least 30 degrees with the horizontal if the plastic flexural strength, M_p , of the remaining tapered section is adequate to resist the shear force attributed to that arm of the shearhead.

422.6.9.4 Compression flanges of steel shapes shall be within $0.3d$ of the compression surface of the slab.

422.6.9.5 The ratio α_v between the flexural stiffness of each shearhead arm and that of the surrounding composite cracked slab section of width $(c_2 + d)$ shall be at least 0.15.

422.6.9.6 For each arm of the shearhead, M_p shall satisfy:

$$M_p \geq \frac{V_u}{2\phi n} \left[h_v + \alpha_v \left(\ell_v - \frac{c_1}{2} \right) \right] \quad (422.6.9.6)$$

where ϕ corresponds to tension-controlled members, n is the number of shearhead arms, and ℓ_v is the minimum length of each shearhead arm required to satisfy Sections 422.6.9.8 and 422.6.9.10.

422.6.9.7 Nominal flexural strength contributed to each slab column strip by a shearhead, M_v , shall satisfy:

$$M_v \leq \frac{\phi \alpha_v V_u}{2n} \left(\ell_v - \frac{c_1}{2} \right) \quad (422.6.9.7)$$

where ϕ corresponds to tension-controlled members. However, M_v shall not exceed the least of (a) through (c):

a. 30 percent of M_u in each slab column strip;

b. Change in M_u in column strip over the length ℓ_v stirrups;

c. M_p as given in Section 422.6.9.6.

422.6.9.8 The critical section for shear shall be perpendicular to the plane of the slab and shall cross each shearhead arm at a distance $(3/4)[\ell_v - (c_1/2)]$ from the column face. This critical section shall be located so that b_o is a minimum, but need not be closer than $d/2$ to the edges of the supporting column.

422.6.9.9 If an opening is located within a column strip or closer than $10h$ from a column in slabs with shearheads, the ineffective portion of b_o shall be one-half of that given in Section 422.6.4.3.

422.6.9.10 Factored shear stress due to vertical loads shall not be greater than $0.33\phi\sqrt{f'_c}$ on the critical section given in Section 422.6.9.8 and shall not be greater than $0.58\phi\sqrt{f'_c}$ on the critical section closest to the column given in Section 422.6.4.1(a).

422.6.9.11 Where transfer of moment is considered, the shearhead shall have adequate anchorage to transmit M_p to the column.

422.6.9.12 Where transfer of moment is considered, the sum of factored shear stresses due to vertical load acting on the critical section given in Section 422.6.9.8 and the shear stresses resulting from factored moment transferred by eccentricity of shear about the centroid of the critical section closest to the column given in Section 422.6.4.1(a) shall not exceed $0.33\phi\lambda\sqrt{f'_c}$.

422.7 Torsional Strength

422.7.1 General

422.7.1.1 This section shall apply to members if $T_u \geq \phi T_{th}$, where ϕ is given in Section 421 and threshold torsion, T_{th} , is given in Section 422.7.4. If $T_u < \phi T_{th}$, it shall be permitted to neglect torsional effects.

422.7.1.2 Nominal torsional strength shall be calculated in accordance with Section 422.7.6.

422.7.1.3 For calculation of T_{th} and T_{cr} , λ shall be in accordance with Section 419.2.4.

422.7.2 Limiting Material Strengths

422.7.2.1 The value of λ used to calculate T_{th} and T_{cr} shall not exceed 8.3 MPa.

422.7.2.2 The values of f_y and f_{yt} for longitudinal and transverse torsional reinforcement shall not exceed the limits in Section 420.2.2.4.

422.7.3 Factored Design Torsion

422.7.3.1 If $T_u \geq \phi T_{cr}$ and T_u is required to maintain equilibrium, the member shall be designed to resist T_u .

422.7.3.2 In a statically indeterminate structure where $T_u \geq \phi T_{cr}$ and a reduction of T_u in a member can occur due to redistribution of internal forces after torsional cracking, T_u to ϕT_{cr} , where the cracking torsion, T_{cr} , is calculated in accordance with Section 422.7.5.

422.7.3.3 If T_u is redistributed in accordance with Section 422.7.3.2, the factored moments and shears used for design of the adjoining members shall be in equilibrium with the reduced torsion.

422.7.4 Threshold Torsion

422.7.4.1 Threshold torsion, T_{th} , shall be calculated in accordance with Table 422.7.4.1(a) for solid cross sections and Table 422.7.4.1(b) for hollow cross sections, where N_u is positive for compression and negative for tension.

Table 422.7.4.1(a)
Threshold Torsion for Solid Cross Sections

Type of member	T_{th}	
Non-prestressed member	$0.083\lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{p_{cp}} \right)$	(a)
Prestressed member	$0.083\lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{p_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{0.33\lambda \sqrt{f'_c}}}$	(b)
Non-prestressed member subjected to axial force	$0.083\lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{p_{cp}} \right) \sqrt{1 + \frac{N_u}{0.33A_g \lambda \sqrt{f'_c}}}$	(c)

Table 422.7.4.1(b)
Threshold Torsion for Hollow Cross Sections

Type of member	T_{th}	
Non-prestressed member	$0.083\lambda \sqrt{f'_c} \left(\frac{A_g^2}{p_{cp}} \right)$	(a)
Prestressed member	$0.083\lambda \sqrt{f'_c} \left(\frac{A_g^2}{p_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{0.33\lambda \sqrt{f'_c}}}$	(b)
Non-prestressed member subjected to axial force	$0.083\lambda \sqrt{f'_c} \left(\frac{A_g^2}{p_{cp}} \right) \sqrt{1 + \frac{N_u}{0.33A_g \lambda \sqrt{f'_c}}}$	(c)

422.7.5 Cracking Torsion

422.7.5.1 Cracking torsion, T_{cr} shall be calculated in accordance with Table 422.7.5.1 for solid and hollow cross sections, where N_u is positive for compression and negative for tension.

Table 422.7.5.1 Cracking Torsion

Type of member	T_{th}	
Non-prestressed member	$0.33\lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{p_{cp}} \right)$	(a)
Prestressed member	$0.33\lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{p_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{0.33\lambda \sqrt{f'_c}}}$	(b)
Non-prestressed member subjected to axial force	$0.33\lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{p_{cp}} \right) \sqrt{1 + \frac{N_u}{0.33A_g \lambda \sqrt{f'_c}}}$	(c)

422.7.6 Torsional Strength

422.7.6.1 For non-prestressed and prestressed members, T_n shall be the lesser of (a) and (b):

$$a. \quad T_n = \frac{2A_o A_t f_{yt}}{s} \cot \theta \quad (422.7.6.1a)$$

$$b. \quad T_n = \frac{2A_o A_\ell f_y}{p_h} \cot \theta \quad (422.7.6.1b)$$

where A_o shall be determined by analysis, θ shall not be taken less than 30 degrees nor greater than 60 degrees, A_t is the area of one leg of a closed stirrup resisting torsion, A_ℓ is the area of longitudinal torsional reinforcement, and p_h is the perimeter of the centerline of the outermost closed stirrup.

422.7.6.1.1 In Eq. 422.7.6.1a and 422.7.6.1b, it shall be permitted to take A_o equal to $0.85A_{oh}$.

422.7.6.1.2 In Eq. 422.7.6.1a and 422.7.6.1b, it shall be permitted to take θ equal to (a) or (b):

- 45 degrees for non-prestressed members or members with $A_{ps}f_{se} < 0.4(A_{ps}f_{pu} + A_s f_y)$;
- 37.5 degrees for prestressed members with $A_{ps}f_{se} \geq 0.4(A_{ps}f_{pu} + A_s f_y)$.

422.7.7 Cross-Sectional Limits

422.7.7.1 Cross-sectional dimensions shall be selected such that (a) or (b) is satisfied:

a. For solid sections

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u p_h}{1.7 A_{oh}^2}\right)^2} \leq \phi \left(\frac{V_c}{b_w d} + 0.66 \sqrt{f'_c}\right) \quad (422.7.7.1a)$$

b. For hollow sections

$$\left(\frac{V_u}{b_w d}\right) + \left(\frac{T_u p_h}{1.7 A_{oh}^2}\right) \leq \phi \left(\frac{V_c}{b_w d} + 0.66 \sqrt{f'_c}\right) \quad (422.7.7.1b)$$

422.7.7.1.1 For prestressed members, the value of d used in Section 422.7.7.1 need not be taken less than $0.8h$.

422.7.7.1.2 For hollow sections where the wall thickness varies around the perimeter, Eq. 422.7.7.1b shall be evaluated at the location where the term

$$\left(\frac{V_u}{b_w d}\right) + \left(\frac{T_u p_h}{1.7 A_{oh}^2}\right)$$

is a maximum.

422.7.7.2 For hollow sections where the wall thickness is less than A_{oh}/p_h , the term $(T_u p_h / A_{oh}^2)$ in Eq. 422.7.7.1b shall be taken as $(T_u / 1.7 A_{oh} t)$, where t is the thickness of the wall of the hollow section at the location where the stresses are being checked.

422.8 Bearing

422.8.1 General

422.8.1.1 This section shall apply to the calculation of bearing strength of concrete members.

422.8.1.2 Bearing strength provisions in this section shall not apply to post-tensioned anchorage zones or strut-and-tie models.

422.8.2 Required Strength

422.8.2.1 Factored compressive force transferred through bearing shall be calculated in accordance with the factored load combinations defined in Section 405 and analysis procedures defined in Section 406.

422.8.3 Design Strength

422.8.3.1 Design bearing strength shall satisfy:

$$\phi B_n \geq B_u \quad (422.8.3.1)$$

for each applicable factored load combination.

422.8.3.2 Nominal bearing strength, B_n , shall be calculated in accordance with Table 422.8.3.2, where A_1 is the loaded area and A_2 is the area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having its upper base equal to the loaded area. The sides of the pyramid, cone, or tapered wedge shall be sloped 1 vertical to 2 horizontal.

Table 422.8.3.2
Nominal Bearing Strength

Geometry of bearing area	B_n	
	Lesser of (a) and (b)	
Supporting surface is wider on all sides than the loaded area	$\sqrt{A_2/A_1} (0.85 f'_c A_1)$	(a)
	$2(0.85 f'_c A_1)$	(b)
Other cases	$0.85 f'_c A_1$	(c)

422.9 Shear Friction

422.9.1 General

422.9.1.1 This section shall apply where it is appropriate to consider shear transfer across any given plane, such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

422.9.1.2 The required area of shear-friction reinforcement across the assumed shear plane, A_{vf} , shall be calculated in accordance with Section 422.9.4. Alternatively, it shall be permitted to use shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests.

422.9.1.3 The value of f_y used to calculate V_n for shear friction shall not exceed the limit in Section 420.2.2.4.

422.9.1.4 Surface preparation of the shear plane assumed for design shall be specified in the construction documents.

422.9.2 Required Strength

422.9.2.1 Factored forces across the assumed shear plane shall be calculated in accordance with the factored load combinations defined in Section 405 and analysis procedures defined in Section 406.

422.9.3 Design Strength

422.9.3.1 Design shear strength across the assumed shear plane shall satisfy:

$$\phi V_n \geq V_u \quad (422.9.3.1)$$

for each applicable factored load combination.

422.9.4 Nominal Shear Strength

422.9.4.1 Value of V_n across the assumed shear plane shall be calculated in accordance with Section 422.9.4.2 or 422.9.4.3. V_n shall not exceed the value calculated in accordance with Section 422.9.4.4.

422.9.4.2 If shear-friction reinforcement is perpendicular to the shear plane, nominal shear strength across the assumed shear plane shall be calculated by:

$$V_n = \mu A_{vf} f_y \quad (422.9.4.2)$$

where A_{vf} is the area of reinforcement crossing the assumed shear plane to resist shear and μ is the coefficient of friction in accordance with Table 422.9.4.2.

Table 422.9.4.2 Coefficients of Friction⁽¹⁾

Contact surface condition	Coefficient of friction μ	
Concrete placed monolithically	1.4λ	(a)
Concrete placed against hardened concrete intentionally roughened to a full amplitude of approximately 6 mm.	1.0λ	(b)
Concrete placed against hardened concrete not intentionally roughened	0.6λ	(c)
Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.	0.7λ	(d)

⁽¹⁾ $\lambda = 1.0$ for normal-weight concrete; $\lambda = 0.75$ for all lightweight concrete. Otherwise, λ is calculated based on volumetric proportions of lightweight and normal-weight aggregate as given in Section 419.2.4.1, but shall not exceed 0.85.

422.9.4.3 If shear-friction reinforcement is inclined to the shear plane and the shear force induces tension in the shear-friction reinforcement, nominal shear strength across the assumed shear plane shall be calculated by:

$$V_n = A_{vf} f_y (\mu \sin \alpha + \cos \alpha) \quad (422.9.4.3)$$

where α is the angle between shear-friction reinforcement and assumed shear plane and μ is the coefficient of friction in accordance with Table 422.9.4.2.

422.9.4.4 The value of V_n across the assumed shear plane shall not exceed the limits in Table 422.9.4.4. Where concretes of different strengths are cast against each other, the lesser value of f'_c shall be used in Table 422.9.4.4.

Table 422.9.4.4 Maximum V_n Across the Assumed Shear Plane

Condition	Maximum V_n		
Normal weight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 6 mm.	Least of (a), (b), and (c)	$0.2f'_c A_c$	(a)
		$(3.3 + 0.08f'_c) A_c$	(b)
		$11A_c$	(c)
Other cases	Lesser of (d) and (e)	$0.2f'_c A_c$	(d)
		$5.5A_c$	(e)

422.9.4.5 Permanent net compression across the shear plane shall be permitted to be added to $A_{vf} f_y$, the force in the shear-friction reinforcement, to calculate required A_{vf} .

422.9.4.6 Area of reinforcement required to resist a net factored tension across an assumed shear plane shall be added to the area of reinforcement required for shear friction crossing the assumed shear plane.

422.9.5 Detailing for Shear-friction Reinforcement

422.9.5.1 Reinforcement crossing the shear plane to satisfy Section 422.9.4 shall be anchored to develop f_y on both sides of the shear plane.

SECTION 423

STRUT-AND-TIE MODELS

423.1 Scope

423.1.1 This section shall apply to the design of structural concrete members, or regions of members, where load or geometric discontinuities cause a nonlinear distribution of longitudinal strains within the cross section.

423.1.2 Any structural concrete member, or discontinuity region in a member, shall be permitted to be designed by modeling the member or region as an idealized truss. This truss shall be designed in accordance with the strut-and-tie method in Section 423.

423.2 General

423.2.1 Strut-and-tie models shall consist of struts and ties connected at nodes to form an idealized truss.

423.2.2 Geometry of the idealized truss shall be consistent with the dimensions of the struts, ties, nodal zones, bearing areas, and supports.

423.2.3 Strut-and-tie models shall be capable of transferring all factored loads to supports or adjacent B-regions.

423.2.4 The internal forces in strut-and-tie models shall be in equilibrium with the applied loads and reactions.

423.2.5 Ties shall be permitted to cross struts and other ties.

423.2.6 Struts shall intersect or overlap only at nodes.

423.2.7 The angle between the axes of any strut and any tie entering a single node shall be at least 25 degrees.

423.2.8 Deep beams designed using strut-and-tie models shall satisfy Sections 409.9.2.1, 409.9.3.1, and 409.9.4.

423.2.9 Brackets and corbels with shear span-to-depth ratio $a_v/d < 2.0$ designed using strut-and-tie models shall satisfy Sections 416.5.2, 416.5.6, and Eq. 423.2.9.

$$A_{sc} > 0.04(f'_c/f_y)(b_w d) \quad (423.2.9)$$

423.3 Design Strength

423.3.1 For each applicable factored load combination, design strength of each strut, tie, and nodal zone in a strut-and-tie model shall satisfy $\phi S_n \geq U$, including (a) through (c):

- a. Struts $\phi F_{ns} \geq F_{us}$
- b. Ties $\phi F_{nt} \geq F_{ut}$
- c. Nodal zones $\phi F_{nn} \geq F_{us}$

423.3.2 ϕ shall be in accordance with Section 421.2

423.4 Strength of Struts

423.4.1 The nominal compressive strength of a strut, F_{ns} , shall be calculated by (a) or (b):

- a. Strut without longitudinal reinforcement

$$F_{ns} = f_{ce} A_{cs} \quad (423.4.1a)$$

- b. Strut with longitudinal reinforcement

$$F_{ns} = f_{ce} A_{cs} + A'_s f'_s \quad (423.4.1b)$$

where F_{ns} shall be evaluated at each end of the strut and taken as the lesser value, A_{cs} is the cross-sectional area at the end of the strut under consideration, f_{ce} is given in Section 423.4.3, A'_s is the area of compression reinforcement along the length of the strut, and f'_s is the stress in the compression reinforcement at the nominal axial strength of the strut. It shall be permitted to take f'_s equal to f_y for Grade 280 or Grade 420 reinforcement.

423.4.2 Effective compressive strength of concrete in a strut, f_{ce} , shall be calculated in accordance with Section 423.4.3 or 423.4.4.

423.4.3 Effective compressive strength of concrete in a strut, f_{ce} , shall be calculated by:

$$f_{ce} = 0.85 \beta_s f'_c \quad (423.4.3)$$

where β_s , in accordance with Table 423.4.3, accounts for the effect of cracking and crack-control reinforcement on the effective compressive strength of the concrete.

Table 423.4.3 Strut Coefficient β_s

Strut geometry and location	Reinforcement crossing a strut	β_s	
Struts with uniform cross-sectional area along length	NA	1.0	(a)
Struts located in a region of a member where the width of the compressed concrete at mid-length of the strut can spread laterally (bottle-shaped struts)	Satisfying Section 423.5	0.75	(b)
	Not Satisfying Section 423.5	0.60λ	(c)
Struts located in tension members or the tension zones of members	NA	0.40	(d)
All other cases	NA	0.60λ	(e)

423.4.4 If confining reinforcement is provided along the length of a strut and its effect is documented by tests and analyses, it shall be permitted to use an increased value of f_{ce} when calculating F_{ns} .

423.5 Reinforcement Crossing Bottle-Shaped Struts

423.5.1 For bottle-shaped struts designed using $\beta_s = 0.75$, reinforcement to resist transverse tension resulting from spreading of the compressive force in the strut shall cross the strut axis. It shall be permitted to determine the transverse tension by assuming that the compressive force in a bottle-shaped strut spreads at a slope of 2 parallel to 1 perpendicular to the axis of the strut.

423.5.2 Reinforcement required in Section 423.5.1 shall be developed beyond the extents of the strut in accordance with Section 425.4.

423.5.3 Distributed reinforcement calculated in accordance with Eq. 423.5.3 and crossing the strut axis shall be deemed to satisfy Section 423.5.1, if $f'_s \leq 40\text{MPa}$.

$$\sum \frac{A_{si}}{b_s s_i} \sin \alpha_i \geq 0.003 \quad (423.5.3)$$

where A_{si} is the total area of distributed reinforcement at spacing s_i in the i -th direction of reinforcement crossing a strut at an angle α_i to the axis of a strut, and b_s is the width of the strut.

423.5.3.1 Distributed reinforcement required in Section 423.5.3 shall be placed orthogonally at angles α_1 and

α_2 to the axis of the strut, or in one direction at an angle α_1 to the axis of the strut. Where the reinforcement is placed in only one direction, α_1 shall be at least 40 degrees.

423.6 Strut Reinforcement Detailing

423.6.1 Compression reinforcement in struts shall be parallel to the axis of the strut and enclosed along the length of the strut by closed ties in accordance with Section 423.6.3 or by spirals in accordance with Section 423.6.4.

423.6.2 Compression reinforcement in struts shall be anchored to develop f'_s at the face of the nodal zone, where f'_s is calculated in accordance with Section 423.4.1.

423.6.3 Closed ties enclosing compression reinforcement in struts shall satisfy Section 425.7.2 and this section.

423.6.3.1 Spacing of closed ties, s , along the length of the strut shall not exceed the smallest of (a) through (c):

- Smallest dimension of cross section of strut;
- $48d_b$ of bar or wire used for closed tie reinforcement;
- $16d_b$ of compression reinforcement.

423.6.3.2 The first closed tie shall be located not more than $0.5s$ from the face of the nodal zone at each end of a strut.

423.6.3.3 Closed ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by crossties or the corner of a tie with an included angle of not more than 135 degrees and no longitudinal bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar.

423.6.4 Spirals enclosing compression reinforcement in struts shall satisfy Section 425.7.3.

423.7 Strength of Ties

423.7.1 Tie reinforcement shall be non-prestressed or prestressed.

423.7.2 The nominal tensile strength of a tie, F_{nt} , shall be calculated by:

$$F_{nt} = A_{ts} f_y + A_{tp} (f_{se} + \Delta f_p) \quad (423.7.2)$$

where $(f_{se} + \Delta f_p)$ shall not exceed f_{py} , and A_{tp} is zero for non-prestressed members.

423.7.3 In Eq. 423.7.2, it shall be permitted to take Δf_p equal to 420 MPa for bonded prestressed reinforcement and 70 MPa for unbonded prestressed reinforcement. Higher values of Δf_p shall be permitted if justified by analysis.

423.8 Tie Reinforcement Detailing

423.8.1 The centroidal axis of the tie reinforcement shall coincide with the axis of the tie assumed in the strut-and-tie model.

423.8.2 Tie reinforcement shall be anchored by mechanical devices, post-tensioning anchorage devices, standard hooks, or straight bar development in accordance with Section 423.8.3.

423.8.3 Tie reinforcement shall be developed in accordance with (a) or (b):

- The difference between the tie force on one side of a node and the tie force on the other side shall be developed within the nodal zone.
- At nodal zones anchoring one or more ties, the tie force in each direction shall be developed at the point where the centroid of the reinforcement in the tie leaves the extended nodal zone.

423.9 Strength of Nodal Zones

423.9.1 The nominal compressive strength of a nodal zone, F_{nn} , shall be calculated by:

$$F_{nn} = f_{ce} A_{nz} \quad (423.9.1)$$

where f_{ce} is defined in Section 423.9.2 or 423.9.3 and A_{nz} is given in Section 423.9.4 or 423.9.5.

423.9.2 The effective compressive strength of concrete at a face of a nodal zone, f_{ce} , shall be calculated by:

$$f_{ce} = 0.85 \beta_n f'_c \quad (423.9.2)$$

where β_n shall be in accordance with Table 423.9.2.

Table 423.9.2 Nodal Zone Coefficient β_n

Configuration of nodal zone	β_n	
Nodal zone bounded by struts, bearing areas, or both	1.0	(a)
Nodal zone anchoring one tie	0.80	(b)
Nodal zone anchoring two or more ties	0.60	(c)

423.9.3 If confining reinforcement is provided within the nodal zone and its effect is documented by tests and analyses, it shall be permitted to use an increased value of f_{ce} when calculating F_{nn} .

423.9.4 The area of each face of a nodal zone, A_{nz} , shall be taken as the smaller of (a) and (b):

- Area of the face of the nodal zone perpendicular to the line of action of F_{us} ;
- Area of a section through the nodal zone perpendicular to the line of action of the resultant force on the section.

423.9.5 In a three-dimensional strut-and-tie model, the area of each face of a nodal zone shall be at least that given in Section 423.9.4, and the shape of each face of the nodal zone shall be similar to the shape of the projection of the end of the strut onto the corresponding face of the nodal zone.

SECTION 424 SERVICEABILITY REQUIREMENTS

424.1 Scope

424.1.1 This section shall apply to member design for minimum serviceability, including (a) through (d):

- Section 424.2 Deflections due to service-level gravity loads;
- Section 424.3 Distribution of flexural reinforcement in one-way slabs and beams to control cracking;
- Section 424.4 Shrinkage and temperature reinforcement;
- Section 424.5 Permissible stresses in prestressed flexural members.

424.2 Deflections Due to Service-Level Gravity Loads

424.2.1 Members subjected to flexure shall be designed with adequate stiffness to limit deflections or

deformations that adversely affect strength or serviceability of a structure.

424.2.2 Deflections calculated in accordance with Sections 424.2.3 through 424.2.5 shall not exceed the limits in Table 424.2.2.

424.2.3 Calculation of Immediate Deflections

424.2.3.1 Immediate deflections shall be calculated using methods or formulas for elastic deflections, considering effects of cracking and reinforcement on member stiffness.

424.2.3.2 Effect of variation of cross-sectional properties, such as haunches, shall be considered when calculating deflections.

424.2.3.3 Deflections in two-way slab systems shall be calculated taking into account size and shape of the panel, conditions of support, and nature of restraints at the panel edges.

424.2.3.4 Modulus of elasticity, E_c , shall be permitted to be calculated in accordance with Section 419.2.2.

Table 424.2.2 Maximum Permissible Calculated Deflections

Member	Condition		Deflection to be considered	Deflection limitation
Flat roofs	Not supporting or attached to non-structural elements likely to be damaged by large deflections		Immediate deflection due to maximum of L_r and R	$\ell/180^{[1]}$
Floors			Immediate deflection due to L	$\ell/360$
Roof or floors	Supporting or attached to non-structural elements	likely to be damaged by large deflections	That part of the total deflection occurring after attachment of non-structural elements, which is the sum of the time-dependent deflection due to all sustained loads and the immediate deflection due to any additional live load. ^[2]	$\ell/480^{[3]}$
		not likely to be damaged by large deflections		$\ell/240^{[4]}$

^[1] Limit not intended to safeguard against ponding. Ponding shall be checked by calculations of deflection, including added deflections due to ponded water, and considering time-dependent effects of sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

^[2] Time-dependent deflection shall be calculated in accordance with Section 424.2.4, but shall be permitted to be reduced by amount of deflection calculated to occur before attachment of non-structural elements. This amount shall be calculated on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

^[3] Limit shall be permitted to be exceeded if measures are taken to prevent damage to supported or attached elements.

^[4] Limit shall not exceed tolerance provided for nonstructural elements

424.2.3.5 For non-prestressed members, effective moment of inertia, I_e , shall be calculated by Eq. 424.2.3.5a unless obtained by a more comprehensive analysis, but I_e shall not be greater than I_g .

$$I_e = \left(\frac{M_{cr}}{M_a} \right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \quad (424.2.3.5a)$$

where M_{cr} is calculated by

$$M_{cr} = \frac{f_r I_g}{y_t} \quad (424.2.3.5b)$$

424.2.3.6 For continuous one-way slabs and beams, I_e shall be permitted to be taken as the average of values obtained from Eq. 424.2.3.5a for the critical positive and negative moment sections.

424.2.3.7 For prismatic one-way slabs and beams, I_e shall be permitted to be taken as the value obtained from Eq. 424.2.3.5a at mid-span for simple and continuous spans, and at the support for cantilevers.

424.2.3.8 For prestressed Class U slabs and beams as defined in Section 424.5.2, it shall be permitted to calculate deflections based on I_g .

424.2.3.9 For prestressed Class T and Class C slabs and beams as defined in Section 424.5.2, deflection calculations shall be based on a cracked transformed section analysis. It shall be permitted to base deflection calculations on a bilinear moment-deflection relationship or I_e in accordance with Eq. 424.2.3.5a, where M_{cr} is calculated as:

$$M_{cr} = \frac{(f_r + f_{pe}) I_g}{y_t} \quad (424.2.3.9)$$

424.2.4 Calculation of Time-Dependent Deflections

424.2.4.1 Non-Prestressed Members

424.2.4.1.1 Unless obtained from a more comprehensive analysis, additional time-dependent deflection resulting from creep and shrinkage of flexural members shall be calculated as the product of the immediate deflection caused by sustained load and the factor λ_Δ .

$$\lambda_\Delta = \frac{\xi}{1 + 50\rho'} \quad (424.2.4.1.1)$$

424.2.4.1.2 In Eq. 424.2.4.1.1, ρ' shall be calculated at mid-span for simple and continuous spans, and at the support for cantilevers.

424.2.4.1.3 In Eq. 424.2.4.1.1, values of the time dependent factor for sustained loads, ξ , shall be in accordance with Table 424.2.4.1.3.

Table 424.2.4.1.3
Time-Dependent Factor for Sustained Loads

Sustained load duration, months	Time-dependent factor ξ
3	1.0
6	1.2
12	1.4
60 or more	2.0

424.2.4.2 Prestressed Members

424.2.4.2.1 Additional time-dependent deflection of prestressed concrete members shall be calculated considering stresses in concrete and reinforcement under sustained load, and the effects of creep and shrinkage of concrete and relaxation of prestressed reinforcement.

424.2.5 Calculation of Deflections of Composite Concrete Construction

424.2.5.1 If composite concrete flexural members are shored during construction so that, after removal of temporary supports, the dead load is resisted by the full composite section, it shall be permitted to consider the composite member equivalent to a monolithically cast member for calculation of deflections.

424.2.5.2 If composite concrete flexural members are not shored during construction, the magnitude and duration of load before and after composite action becomes effective shall be considered in calculating time-dependent deflections.

424.2.5.3 Deflections resulting from differential shrinkage of precast and cast-in-place components, and of axial creep effects in prestressed members, shall be considered.

424.3 Distribution of Flexural Reinforcement in One-Way Slabs and Beams

424.3.1 Bonded reinforcement shall be distributed to control flexural cracking in tension zones of non-prestressed and Class C prestressed slabs and beams reinforced for flexure in one direction only.

424.3.2 Spacing of bonded reinforcement closest to the tension face shall not exceed the limits in Table 424.3.2, where cc is the least distance from surface of deformed or prestressed reinforcement to the tension face. Calculated stress in deformed reinforcement, f_s , and calculated change in stress in bonded prestressed reinforcement, Δf_{ps} , shall be in accordance with Sections 424.3.2.1 and 424.3.2.2, respectively.

Table 424.3.2
Maximum Spacing of Bonded
Reinforcement in Non-Prestressed and Class C
Prestressed One-Way Slabs and Beams

Reinforcement type	Maximum spacing s	
Deformed bars or wires	Lesser of:	$380 \left(\frac{280}{f_s} \right) - 2.5c_c$
		$300 \left(\frac{280}{f_s} \right)$
Bonded prestressed reinforcement	Lesser of:	$\left(\frac{2}{3} \right) \left[380 \left(\frac{280}{\Delta f_{ps}} \right) - 2.5c_c \right]$
		$\left(\frac{2}{3} \right) \left[300 \left(\frac{280}{\Delta f_{ps}} \right) \right]$
Combined deformed bars or wires and bonded prestressed reinforcement	Lesser of:	$\left(\frac{5}{6} \right) \left[380 \left(\frac{280}{\Delta f_{ps}} \right) - 2.5c_c \right]$
		$\left(\frac{5}{6} \right) \left[380 \left(\frac{280}{\Delta f_{ps}} \right) \right]$

424.3.2.1 Stress f_s in deformed reinforcement closest to the tension face at service loads shall be calculated based on the unfactored moment, or it shall be permitted to take f_s as $(2/3)f_y$.

424.3.2.2 Change in stress Δf_{ps} in bonded prestressed reinforcement at service loads shall be equal to the calculated stress based on a cracked section analysis minus the decompression stress f_{dc} . It shall be permitted to take f_{dc} equal to the effective stress in the prestressed reinforcement f_{se} . The value of Δf_{ps} shall not exceed 250 MPa. If Δf_{ps} does not exceed 140 MPa, the spacing limits in Table 424.3.2 need not be satisfied.

424.3.3 If there is only one bonded bar, pretensioned strand, or bonded tendon nearest to the extreme tension face, the width of the extreme tension face shall not exceed s determined in accordance with Table 424.3.2.

424.3.4 If flanges of T-beams are in tension, part of the bonded flexural tension reinforcement shall be distributed over an effective flange width as defined in accordance with Section 406.3.2, but not wider than $\ell_n/10$. If the effective flange width exceeds $\ell_n/10$, additional bonded longitudinal reinforcement shall be provided in the outer portions of the flange.

424.3.5 The spacing of bonded flexural reinforcement in non-prestressed and Class C prestressed one-way slabs and beams subject to fatigue, designed to be watertight, or exposed to corrosive environments, shall be selected based on investigations and precautions specific to those conditions and shall not exceed the limits of Section 424.3.2.

424.4 Shrinkage and Temperature Reinforcement

424.4.1 Reinforcement to resist shrinkage and temperature stresses shall be provided in one-way slabs in the direction perpendicular to the flexural reinforcement in accordance with Section 424.4.3 or 424.4.4.

424.4.2 If shrinkage and temperature movements are restrained, the effects of T shall be considered in accordance with Section 405.3.6.

424.4.3 Non-Prestressed Reinforcement

424.4.3.1 Deformed reinforcement to resist shrinkage and temperature stresses shall conform to Table 420.2.2.4(a) and shall be in accordance with Sections 424.4.3.2 through 424.4.3.5.

424.4.3.2 The ratio of deformed shrinkage and temperature reinforcement area to gross concrete area shall satisfy the limits in Table 424.4.3.2.

Table 424.4.3.2
Minimum Ratios of Deformed Shrinkage and
Temperature Reinforcement Area to Gross Concrete
Area

Reinforcement type	f_y , MPa	Minimum reinforcement ratio	
Deformed bars	<420	0.0020	
Deformed bars or welded wire reinforcement	≥ 420	Greater of:	$\frac{0.0018 \times 420}{f_y}$
			0.0014

424.4.3.3 The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of **5h** and 450 mm.

424.4.3.4 At all sections where required, deformed reinforcement used to resist shrinkage and temperature stresses shall develop f_y in tension.

424.4.3.5 For one-way precast slabs and one-way precast, prestressed wall panels, shrinkage and temperature reinforcement is not required in the direction perpendicular to the flexural reinforcement if (a) through (c) are satisfied.

- a. Precast members are not wider than 3.7 m;
- b. Precast members are not mechanically connected to cause restraint in the transverse direction;
- c. Reinforcement is not required to resist transverse flexural stresses.

424.4.4 Prestressed Reinforcement

424.4.4.1 Prestressed reinforcement to resist shrinkage and temperature stresses shall conform to Table 420.3.2.2 and the effective prestress after losses shall provide an average compressive stress of at least 0.7 MPa on gross concrete area.

424.5 Permissible Stresses in Prestressed Concrete Flexural Members

424.5.1 General

424.5.1.1 Concrete stresses in prestressed flexural members shall be limited in accordance with Section 424.5.2 through 424.5.4 unless it is shown by test or analysis that performance will not be impaired.

424.5.1.2 For calculation of stresses at transfer of prestress, at service loads, and at cracking loads, elastic theory shall be used with assumptions (a) and (b):

- a. Strains vary linearly with distance from neutral axis in accordance with Section 422.2.1;
- b. At cracked sections, concrete resists no tension.

424.5.2 Classification of Prestressed Flexural Members

424.5.2.1 Prestressed flexural members shall be classified as Class U, T, or C in accordance with Table 424.5.2.1, based on the extreme fiber stress in tension f_t in the precompressed tensile zone calculated at service loads assuming an uncracked section.

Table 424.5.2.1
Classification of Prestressed
Flexural Members Based on f_t

Assumed Behavior	Class	Limits of f_t
Uncracked	U ⁽¹⁾	$f_t \leq 0.62\sqrt{f'_c}$
Transition between uncracked and cracked	T	$0.62\sqrt{f'_c} < f_t \leq \sqrt{f'_c}$
Cracked	C	$f_t > \sqrt{f'_c}$

⁽¹⁾ Prestressed two-way slabs shall be designed as Class U with $f_t \leq 0.5\sqrt{f'_c}$

424.5.2.2 For Class U and T members, stresses at service loads shall be permitted to be calculated using the uncracked section.

424.5.2.3 For Class C members, stresses at service loads shall be calculated using the cracked transformed section.

424.5.3 Permissible Concrete Stresses at Transfer of Prestress

424.5.3.1 Calculated extreme concrete fiber stress in compression immediately after transfer of prestress, but before time-dependent prestress losses, shall not exceed the limits in Table 424.5.3.1.

Table 424.5.3.1
Concrete Compressive Stress Limits
Immediately After Transfer of Prestress

Location	Concrete Compressive Stress Limits
End of simply-supported members	$0.70f'_{ci}$
All other locations	$0.60f'_{ci}$

424.5.3.2 Calculated extreme concrete fiber stress in tension immediately after transfer of prestress, but before time-dependent prestress losses, shall not exceed the limits in Table 424.5.3.2, unless permitted by Section 424.5.3.2.1.

Table 424.5.3.2
Concrete Tensile Stress Limits Immediately After
Transfer of Prestress, Without
Additional Bonded Reinforcement in Tension Zone

Location	Concrete Compressive Stress Limits
End of simply-supported members	$0.5 \sqrt{f'_{ci}}$
All other locations	$0.25 \sqrt{f'_{ci}}$

424.5.3.2.1 The limits in Table 424.5.3.2 shall be permitted to be exceeded where additional bonded reinforcement in the tension zone resists the total tensile force in the concrete calculated with the assumption of an uncracked section.

424.5.4 Permissible Concrete Compressive Stresses at Service Loads

424.5.4.1 For Class U and T members, the calculated extreme concrete fiber stress in compression at service loads, after allowance for all prestress losses, shall not exceed the limits in Table 424.5.4.1.

Table 424.5.4.1
Concrete Compressive Stress Limits at Service Loads

Location	Concrete Compressive Stress Limits
Prestress plus sustained load	$0.45 f'_{ci}$
Prestress plus total load	$0.60 f'_{ci}$

SECTION 425 REINFORCEMENT DETAILS

425.1 Scope

425.1.1 This section shall apply to reinforcement details, including:

- Minimum spacing;
- Standard hooks, seismic hooks, and crossies;
- Development of reinforcement;
- Splices;
- Bundled reinforcement;
- Transverse reinforcement;
- Post-tensioning anchorages and couplers.

425.1.2 The provisions of Section 425.9 shall apply to anchorage zones for post-tensioned tendons.

425.2 Minimum Spacing of Reinforcement

425.2.1 For parallel non-prestressed reinforcement in a horizontal layer, clear spacing shall be at least the greatest of 50 mm, d_b , and $(4/3)d_{agg}$.

425.2.2 For parallel non-prestressed reinforcement placed in two or more horizontal layers, reinforcement in the upper layers shall be placed directly above reinforcement in the bottom layer with a clear spacing between layers of at least 25 mm.

425.2.3 For longitudinal reinforcement in columns, pedestals, struts, and boundary elements in walls, clear spacing between bars shall be at least the greatest of 40 mm, $1.5d_b$, and $(4/3)d_{agg}$.

Table 425.2.4
Minimum Center-to-Center Spacing of Pretensioned Strands at Ends of Members

f'_c , MPa	Nominal strand diameter, mm.	Minimum s
< 28	All	$4d_b$
	<12 mm	$4d_b$
≥ 28	12 mm	45 mm
	15 mm	50 mm

425.2.4 For pretensioned strands at ends of a member, minimum center-to-center spacing s shall be the greater of the value in Table 425.2.4, and $[(4/3)d_{agg} + d_b]$.

425.2.5 For pretensioned wire at ends of a member, minimum center-to-center spacing s shall be the greater of $5d_b$ and $[(4/3)d_{agg} + d_b]$.

425.3 Standard Hooks, Seismic Hooks, Crossties, and Minimum Inside Bend Diameters

425.3.1 Standard hooks for the development of deformed bars in tension shall conform to Table 425.3.1.

425.3.2 Minimum inside bend diameters for bars used as transverse reinforcement and standard hooks for bars used to anchor stirrups, ties, hoops, and spirals shall conform to Table 425.3.2. Standard hooks shall enclose longitudinal reinforcement.

425.3.3 Minimum inside bend diameters for welded wire reinforcement used as stirrups or ties shall not be less than $4d_b$ for deformed wire larger than D6 and $2d_b$ for all other wires. Bends with inside diameter of less than $8d_b$ shall not be less than $4d_b$ from nearest welded intersection.

425.3.4 Seismic hooks used to anchor stirrups, ties, hoops, and crossties shall be in accordance with (a) and (b):

- Minimum bend of 90 degrees for circular hoops and 135 degrees for all other hoops;
- Hook shall engage longitudinal reinforcement and the extension shall project into the interior of the stirrup or hoop.

425.3.5 Crossties shall be in accordance with (a) through (e):

- Crosstie shall be continuous between ends;
- There shall be a seismic hook at one end;
- There shall be a standard hook at other end with minimum bend of 90 degrees;
- Hooks shall engage peripheral longitudinal bars;
- 90-degree hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end, unless crossties satisfy Section 418.6.4.3 or 425.7.1.6.1.

Table 425.3.1
Standard Hook Geometry for Development of Deformed Bars in Tension

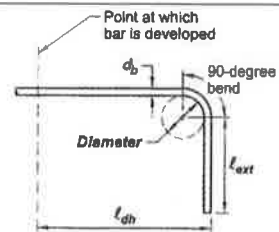
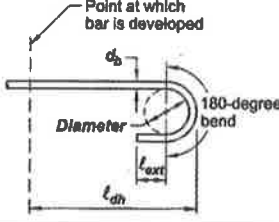
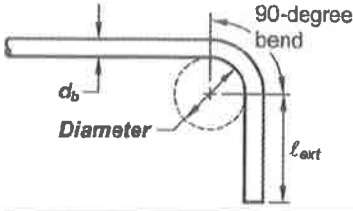
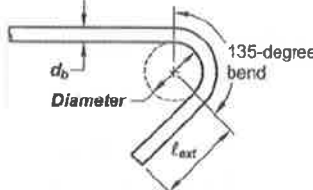
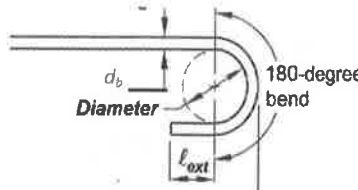
Type of standard hook	Bar size	Minimum inside bend diameter mm.	Straight extension ⁽¹⁾ ℓ_{ext} mm.	Type of standard hook
90-degree hook	10 mm ϕ through 25 mm ϕ	$6d_b$	$12d_b$	
	28 mm ϕ through 36 mm ϕ	$8d_b$		
	40 mm ϕ and 58 mm ϕ	$10d_b$		
180-degree hook	10 mm ϕ through 25 mm ϕ	$6d_b$	Greater of $4d_b$ and 65 mm	
	28 mm ϕ through 36 mm ϕ	$8d_b$		
	40 mm ϕ and 58 mm ϕ	$10d_b$		

Table 425.3.2
Minimum Inside Bend Diameters and Standard Hook Geometry for Stirrups, Ties, and Hoops

Type of standard hook	Bar size	Minimum inside bend diameter mm.	Straight extension ^[1] ℓ_{ext} mm.	Type of standard hook
90-degree hook	10 mm ϕ through 16 mm ϕ	$4d_b$	Greater of $46d_b$ and 75 mm	
	20 mm ϕ through 25 mm ϕ	$6d_b$	$12d_b$	
135-degree hook	10 mm ϕ through 16 mm ϕ	$4d_b$	Greater of $6d_b$ and 75 mm	
	20 mm ϕ through 25 mm ϕ	$6d_b$		
180-degree hook	10 mm ϕ through 16 mm ϕ	$4d_b$	Greater of $6d_b$ and 65 mm	
	20 mm ϕ through 25 mm ϕ	$6d_b$		

^[1] A standard hook for stirrups, ties, and hoops includes the specific inside bend diameter and straight extension length. It shall be permitted to use a longer straight extension at the end of a hook. A longer extension shall not be considered to increase the anchorage capacity of the hook.

425.4 Development of Reinforcement

425.4.1 General

425.4.1.1 Calculated tension or compression in reinforcement at each section of a member shall be developed on each side of that section by embedment length, hook, headed deformed bar, mechanical device, or a combination thereof.

425.4.1.2 Hooks and heads shall not be used to develop bars in compression.

425.4.1.3 Development lengths do not require a strength reduction factor ϕ .

425.4.1.4 The values of $\sqrt{f'_c}$ used to calculate development length shall not exceed 8.3 MPa.

425.4.2 Development of Deformed Bars and Deformed Wires in Tension

425.4.2.1 Development length ℓ_d for deformed bars and deformed wires in tension shall be the greater of (a) and (b):

- Length calculated in accordance with Section 425.4.2.2 or 425.4.2.3 using the applicable modification factors of Section 425.4.2.4;
- 300 mm.

425.4.2.2 For deformed bars or deformed wires, ℓ_d shall be calculated in accordance with Table 425.4.2.2.

Table 425.4.2.2
Development Length for Deformed Bars and Deformed Wires in Tension

Spacing and cover	20 mm ϕ and smaller bars and deformed wires	25 mm ϕ and larger bars
Clear spacing of bars or wires being developed or lap spliced not less than d_b , clear cover at least d_b , and stirrups or ties throughout d_b not less than the Code minimum, or Clear spacing of bars or wires being developed or lap spliced at least $2d_b$ and clear cover at least d_b	$\left(\frac{f_y \Psi_t \Psi_e}{2.1 \lambda \sqrt{f'_c}} \right) d_b$	$\left(\frac{f_y \Psi_t \Psi_e}{1.7 \lambda \sqrt{f'_c}} \right) d_b$
Other cases	$\left(\frac{f_y \Psi_t \Psi_e}{1.4 \lambda \sqrt{f'_c}} \right) d_b$	$\left(\frac{f_y \Psi_t \Psi_e}{1.1 \lambda \sqrt{f'_c}} \right) d_b$

425.4.2.3 For deformed bars or deformed wires, ℓ_d shall be calculated by:

$$\ell_d = \left(\frac{1}{1.1} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\Psi_t \Psi_e \Psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \quad (425.4.2.3a)$$

in which the confinement term $(c_b + K_{tr})/d_b$ shall not exceed 2.5, and

$$K_{tr} = \frac{40 A_{tr}}{sn} \quad (425.4.2.3b)$$

where n is the number of bars or wires being developed or lap spliced along the plane of splitting. It shall be permitted to use $K_{tr} = 0$ as a design simplification even if transverse reinforcement is present.

425.4.2.4 For the calculation of ℓ_d , modification factors shall be in accordance with Table 425.4.2.4.

425.4.3 Development of Standard Hooks in Tension

425.4.3.1 Development length ℓ_{dh} for deformed bars in tension terminating in a standard hook shall be the greater of (a) through (c):

a. $\left(\frac{f_y \Psi_e \Psi_c \Psi_r}{4.17 \lambda \sqrt{f'_c}} \right) d_b$ with Ψ_e , Ψ_c , Ψ_r , and λ given in Section 425.4.3.2;

b. $8d_b$;

c. 150 mm,

Table 425.4.2.4
Modification Factors for Development of Deformed Bars and Deformed Wires in Tension

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Lightweight concrete, where f_{ct} is specified	In accordance with Section 419.2.4.3
	Normal-weight concrete	1.0
Epoxy ⁽¹⁾ Ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement with clear cover less than $3d_b$ or clear spacing less than $6d_b$	1.5
	Epoxy-coated or zinc and epoxy dual-coated reinforcement for all other conditions	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Size Ψ_s	25 mm ϕ and larger bars	1.0
	20 mm ϕ and smaller bars and deformed wires	0.8
Casting position ⁽¹⁾ Ψ_t	More than 300 mm. of fresh concrete placed below horizontal reinforcement	1.3
	Other	1.0

⁽¹⁾ The product $\Psi_t \Psi_e$ need not exceed 1.7.

425.4.3.2 For the calculation of ℓ_{dh} , modification factors shall be in accordance with Table 425.4.3.2. Factors Ψ_c and Ψ_r shall be permitted to be taken as 1.0. At discontinuous ends of members Section 425.4.3.3 shall apply.

Table 425.4.3.2
Modification Factors for Development
of Hooked Bars in Tension

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Lightweight concrete, where f_{ct} is specified	In accordance with Section 419.2.4.3
	Normal-weight concrete	1.0
Epoxy Ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Cover Ψ_c	For 36 mm ϕ bar and smaller hooks with side cover (normal to plane of hook) ≥ 65 mm. and for 90-degree hook with cover on bar extension beyond hook ≥ 50 mm.	0.7
	Other	1.0
Confining reinforcement ^[2] Ψ_r	For 90-degree hooks of 36 mm ϕ and smaller bars 1. enclosed along ℓ_{dh} within ties or stirrups ^[1] perpendicular to ℓ_{dh} at $s \leq 3d_b$ or 2. enclosed along the bar extension beyond hook including the bend within ties or stirrups ^[1] perpendicular to ℓ_{ext} at $s \leq 3d_b$	0.8
	Other	1.0

^[1] The first tie or stirrup shall enclose the bent portion of the hook within $2d_b$ of the outside of the bend.

^[2] d_b is the nominal diameter of the hooked bar.

425.4.3.3 For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 65mm, (a) through (c) shall be satisfied:

- The hook shall be enclosed along ℓ_{dh} within ties or stirrups perpendicular to ℓ_{dh} at $s \leq 3d_b$;
- The first tie or stirrup shall enclose the bent portion of the hook within $2d_b$ of the outside of the bend ;
- Ψ_r shall be taken as 1.0 in calculating ℓ_{dh} in accordance with Section 425.4.3.1(a).

where d_b is the nominal diameter of the hooked bar.

425.4.4 Development of Headed Deformed Bars in Tension

425.4.4.1 Use of heads to develop deformed bars in tension shall be permitted if conditions (a) through (g) are satisfied:

- Bar shall conform to Section 420.2.1.3;
- Bar f_y shall not exceed 420 KPa;
- Bar size shall not exceed 36 mm ϕ bar;
- Net bearing area of head A_{brg} shall be at least $4A_b$;
- Concrete shall be normal-weight;
- Clear cover for bar shall be at least $2d_b$;
- Clear spacing between bars shall be at least $4d_b$.

425.4.4.2 Development length ℓ_{dt} for headed deformed bars in tension shall be the greatest of (a) through (c):

- $\left(\frac{0.19f_y\Psi_e}{\sqrt{f'_c}} \right) d_b$, with Ψ_e , given in

Section 425.4.4.3 and value of f'_c shall not exceed 40 MPa;

- $8d_b$;
- 150 mm.

425.4.4.3 Modification factor Ψ_e in Section 425.4.4.2(a) shall be 1.2 for epoxy-coated or zinc and epoxy dual-coated bars and 1.0 for uncoated or zinc-coated (galvanized) bars.

425.4.5 Development of Mechanically Anchored Deformed Bars in Tension

425.4.5.1 Any mechanical attachment or device capable of developing f_y of deformed bars shall be permitted, provided it is approved by the building official in accordance with Section 401.10. Development of deformed bars shall be permitted to consist of a combination of mechanical anchorage plus additional embedment length of the deformed bars between the critical section and the mechanical attachment or device.

425.4.6 Development of Welded Deformed Wire Reinforcement in Tension

425.4.6.1 Development length ℓ_d for welded deformed wire reinforcement in tension measured from the critical section to the end of wire shall be the greater of (a) and (b), where wires in the direction of the development length shall all be deformed MD200 or smaller.

- Length calculated in accordance with Section 425.4.6.2;
- 200 mm.

425.4.6.2 For welded deformed wire reinforcement, ℓ_d shall be calculated from Section 425.4.2.2 or 425.4.2.3, times welded deformed wire reinforcement factor Ψ_w from Section 425.4.6.3 or 425.4.6.4. For epoxy-coated welded deformed wire reinforcement meeting Section 425.4.6.3, it shall be permitted to use $\Psi_e = 1.0$ in Section 425.4.2.2 or 425.4.2.3.

425.4.6.3 For welded deformed wire reinforcement with at least one cross wire within ℓ_d that is at least 50 mm from the critical section, Ψ_w shall be the greater of (a) and (b), and need not exceed 1.0:

- $\left(\frac{f_y - 240}{f_y} \right)$
- $\left(\frac{5d_b}{s} \right)$

where s is the spacing between the wires to be developed.

425.4.6.4 For welded deformed wire reinforcement with no cross wires within ℓ_d or with a single cross wire less than 50 mm from the critical section, Ψ_w shall be taken as 1.0.

425.4.6.5 Where any plain wires, or deformed wires larger than MD200, are present in the welded deformed wire reinforcement in the direction of the development length, the reinforcement shall be developed in accordance with Section 425.4.7.

425.4.6.6 Zinc-coated (galvanized) welded deformed wire reinforcement shall be developed in accordance with Section 425.4.7.

425.4.7 Development of Welded Plain Wire Reinforcement in Tension

425.4.7.1 Development length ℓ_d for welded plain wire reinforcement in tension measured from the critical

section to the outermost crosswire shall be the greater of (a) and (b) and shall require a minimum of two cross wires within ℓ_d .

- Length calculated in accordance with Section 425.4.7.2;
- 150 mm.

425.4.7.2 ℓ_d shall be the greater of (a) and (b):

- spacing of cross wires +50 mm;
- $3.3 \left(\frac{f_y}{\lambda \sqrt{f'_c}} \right) \left(\frac{A_b}{s} \right)$, where s is the spacing between the wires to be developed, and λ is given in Table 425.4.2.4.

425.4.8 Development of Pretensioned Seven-Wire Strands in Tension

425.4.8.1 Development length ℓ_d of pretensioned seven-wire strands in tension shall be in accordance with (a) and (b):

- $\ell_d = \left(\frac{f_{se}}{21} \right) d_b + \left(\frac{f_{ps} - f_{se}}{7} \right) d_b$ (425.4.8.1)
- If bonding of a strand does not extend to end of member, and design includes tension at service loads in the pre-compressed tensile zone, ℓ_d calculated by Eq. 425.4.8.1 shall be doubled.

425.4.8.2 Seven-wire strand shall be bonded at least ℓ_d beyond the critical section except as provided in Section 425.4.8.3.

425.4.8.3 Embedment less than ℓ_d shall be permitted at a section of a member provided the design strand stress at that section does not exceed values obtained from the bilinear relationship defined by Eq. 425.4.8.1.

425.4.9 Development of Deformed Bars and Deformed Wires in Compression

425.4.9.1 Development length ℓ_{dc} for deformed bars and deformed wires in compression shall be the greater of (a) and (b)

- Length calculated in accordance with Section 425.4.9.2;
- 200 mm.

425.4.9.2 ℓ_{dc} shall be the greater of (a) and (b), multiplied by the modification factors of Section 425.4.9.3:

a. $\left(\frac{0.24 f_y \Psi_r}{\lambda \sqrt{f'_c}} \right) d_b$

b. $0.043 f_y \Psi_r d_b$

425.4.9.3 For the calculation of ℓ_{dc} , modification factors shall be in accordance with Table 425.4.9.3, except Ψ_r shall be permitted to be taken as 1.0.

Table 425.4.9.3
Modification Factors for Deformed Bars
and Wires in Compression

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Lightweight concrete, if f_{ct} is specified	In accordance with Section 419.2.4.3
	Normal-weight concrete	1.0
Confining reinforcement Ψ_r	Reinforcement enclosed within (1), (2), (3) or (4): 1. a spiral 2. a circular continuously wound tie with $d_b \geq 6$ mm in. and pitch ≤ 100 mm 3. 6 mm ϕ bar or MD130 wire ties in accordance with Section 425.7.2 spaced ≤ 100 mm on center 4. hoops in accordance with Section 425.7.4 spaced ≤ 100 mm on center	0.75
	Other	1.0

425.4.10 Reduction of Development Length for Excess Reinforcement

425.4.10.1 Reduction of development lengths defined in Sections 425.4.2.1(a), 425.4.3.1(a), 425.4.6.1(a), 425.4.7.1(a), and 425.4.9.1(a) shall be permitted by use of the ratio $(A_{s,provided}/A_{s,required})$, except where prohibited by Section 425.4.10.2. The modified development lengths shall not be less than the respective minimums specified in Sections 425.4.2.1(b), 425.4.3.1(b), 425.4.3.1(c), 425.4.6.1(b), 425.4.7.1(b), and 425.4.9.1(b).

425.4.10.2 A reduction of development length in accordance with Section 425.4.10.1 is not permitted for (a) through (e).

a. At the face of a non-continuous support;

b. At other locations where anchorage or development for f_y is required;

c. Where bars are required to be continuous;

d. For headed and mechanically anchored deformed reinforcement;

e. In seismic-force-resisting systems in structures assigned to Seismic zone 4.

425.5 Splices

425.5.1 General

425.5.1.1 Lap splices shall not be permitted for bars larger than 36 mm ϕ , except as provided in Section 425.5.5.3.

425.5.1.2 For contact lap splices, minimum clear spacing between the contact lap splice and adjacent splices or bars shall be in accordance with the requirements for individual bars in Section 425.2.1.

425.5.1.3 For non-contact splices in flexural members, the transverse center-to-center spacing of spliced bars shall not exceed the lesser of one-fifth the required lap splice length and 150 mm.

425.5.1.4 Reduction of development length in accordance with Section 425.4.10.1 is not permitted in calculating lap splice lengths.

425.5.1.5 Lap splices of bundled bars shall be in accordance with Section 425.6.1.7.

425.5.2 Lap Splice Lengths of Deformed Bars and Deformed Wires in Tension

425.5.2.1 Tension lap splice length ℓ_{st} for deformed bars and deformed wires in tension shall be in accordance with Table 425.5.2.1, where ℓ_d shall be in accordance with Section 425.4.2.1(a).

Table 425.5.2.1
Lap Splice Lengths of Deformed Bars and Deformed Wires in Tension

$A_{s,provided}/A_{s,required}$ ^[1] over length of splice	Maximum percent of A_s spliced within required lap length	Splice Type	ℓ_{st}	
≥ 2.0	50	Class A	Greater of:	$1.0\ell_d$ and 300 mm
	100	Class B	Greater of:	$1.3\ell_d$ and 300 mm
< 2.0	All cases	Class B		

^[1] Ratio of area of reinforcement provided to area of reinforcement required by analysis at splice location.

425.5.2.2 If bars of different size are lap spliced in tension, ℓ_{st} shall be the greater of ℓ_d of the larger bar and ℓ_{st} of the smaller bar.

425.5.3 Lap splice lengths of welded deformed wire reinforcement in tension

425.5.3.1 Tension lap splice length ℓ_{st} of welded deformed wire reinforcement in tension with cross wires within the lap splice length shall be the greater of $1.3\ell_d$ and 200 mm, where ℓ_d is calculated in accordance with Section 425.4.6.1(a), provided (a) and (b) are satisfied:

- Overlap between outermost cross wires of each reinforcement sheet shall be at least 50 mm;
- Wires in the direction of the development length shall all be deformed MD200 or smaller in accordance with Section 425.5.2.

425.5.3.1.2 If Section 425.5.3.1(b) is not satisfied, ℓ_{st} shall be calculated in accordance with Section 425.5.4.

425.5.3.1.3 If the welded deformed wire reinforcement is zinc-coated (galvanized), ℓ_{st} shall be calculated in accordance with Section 425.5.4.

425.5.4 Lap Splice Lengths of Welded Plain Wire Reinforcement in Tension

425.5.4.1 Tension lap splice length ℓ_{st} of welded plain wire reinforcement in tension between outermost cross wires of each reinforcement sheet shall be at least the greatest of (a) through (c):

- $s + 50 \text{ mm}$;
- $1.5\ell_d$;
- 150 mm.

where s is the spacing of cross wires and ℓ_d is calculated in accordance with Section 425.4.7.2(b).

425.5.4.2 If $A_{s,provided}/A_{s,required} \geq 2.0$ over the length of the splice, ℓ_{st} measured between outermost cross wires of each reinforcement sheet shall be permitted to be the greater of (a) and (b).

- $1.5\ell_d$;
- 50 mm.

where ℓ_d is calculated by Section 425.4.7.2(b).

425.5.5 Lap Splice Lengths of Deformed Bars in Compression

425.5.5.1 Compression lap splice length ℓ_{sc} of 36 mm ϕ or smaller deformed bars in compression shall be calculated in accordance with (a) or (b):

- For $f_y \leq 420 \text{ MPa}$: ℓ_{sc} is the greater of $0.071f_y d_b$ and 300 mm;
- For $f_y > 420 \text{ MPa}$: ℓ_{sc} is the greater of $(0.13f_y - 24)d_b$ and 300 mm.

For $f'_c < 21 \text{ MPa}$, the length of lap shall be increased by one-third.

425.5.5.2 Compression lap splices shall not be used for bars larger than 36 mm ϕ , except as permitted in Section 425.5.5.3.

425.5.5.3 Compression lap splices of 40 mm ϕ or 58 mm ϕ bars to 36 mm ϕ or smaller bars shall be permitted and shall be in accordance with Section 425.5.5.4.

425.5.5.4 Where bars of different size are lap spliced in compression, ℓ_{sc} shall be the greater of ℓ_{dc} of larger bar calculated in accordance with Section 425.4.9.1 and

ℓ_{sc} of smaller bar calculated in accordance with Section 425.5.5.1 as appropriate.

425.5.6 End-Bearing Splices of Deformed Bars in Compression

425.5.6.1 For bars required for compression only, transmission of compressive stress by end bearing of square-cut ends held in concentric contact by a suitable device shall be permitted.

425.5.6.2 End-bearing splices shall be permitted only in members containing closed stirrups, ties, spirals, or hoops.

425.5.6.3 Bar ends shall terminate in flat surfaces within 1.5 degrees of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly.

425.5.7 Mechanical and Welded Splices of Deformed Bars in Tension or Compression

425.5.7.1 A mechanical or welded splice shall develop in tension or compression, as required, at least $1.25f_y$ of the bar.

425.5.7.2 Welding of reinforcing bars shall conform to Section 426.6.4.

425.5.7.3 Mechanical or welded splices need not be staggered except as required by Section 425.5.7.4.

425.5.7.4 Splices in tension tie members shall be made with a mechanical or welded splice in accordance with Section 425.5.7.1. Splices in adjacent bars shall be staggered at least 750 mm.

425.6 Bundled Reinforcement

425.6.1 Non-Prestressed Reinforcement

425.6.1.1 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four in any one bundle.

425.6.1.2 Bundled bars shall be enclosed within transverse reinforcement. Bundled bars in compression members shall be enclosed by transverse reinforcement at least 12 mm ϕ in size.

425.6.1.3 Bars larger than a 36 mm ϕ shall not be bundled in beams.

425.6.1.4 Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least $40d_b$ stagger.

425.6.1.5 Development length for individual bars within a bundle, in tension or compression, shall be that of the individual bar, increased 20 percent for a three-bar bundle, and 33 percent for a four-bar bundle.

425.6.1.6 A unit of bundled bars shall be treated as a single bar with an area equivalent to that of the bundle and a centroid coinciding with that of the bundle. The diameter of the equivalent bar shall be used for d_b in (a) through (e):

- a. Spacing limitations based on d_b ;
- b. Cover requirements based on d_b ;
- c. Spacing and cover values in Section 425.4.2.2;
- d. Confinement term in Section 425.4.2.3;
- e. Ψ_e factor in Section 425.4.2.4.

425.6.1.7 Lap splices of bars in a bundle shall be based on the lap splice length required for individual bars within the bundle, increased in accordance with Section 425.6.1.5. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced.

425.6.2 Post-tensioning Ducts

425.6.2.1 Bundling of post-tensioning ducts shall be permitted if shown that concrete can be satisfactorily placed and if provision is made to prevent the prestressing steel, when tensioned, from breaking through the duct.

425.7 Transverse Reinforcement

425.7.1 Stirrups

425.7.1.1 Stirrups shall extend as close to the compression and tension surfaces of the member as cover requirements and proximity of other reinforcement permits and shall be anchored at both ends. Where used as shear reinforcement, stirrups shall extend a distance d from extreme compression fiber.

425.7.1.2 Between anchored ends, each bend in the continuous portion of a single or multiple U-stirrup and each bend in a closed stirrup shall enclose a longitudinal bar or strand.

425.7.1.3 Anchorage of deformed bar and wire shall be in accordance with (a), (b), or (c):

- a. For 16 mm ϕ bar and MD200 wire, and smaller, and for 20 mm ϕ through 25 mm ϕ bars with $f_{yt} \leq 280$ MPa, a standard hook around longitudinal reinforcement;
- b. For 20 mm ϕ through 25 mm ϕ bars with $f_{yt} > 280$ MPa, a standard hook around a longitudinal bar plus an embedment between mid-height of the member and the outside end of the hook equal to or greater than $0.17d_b f_{yt} / (\lambda \sqrt{f'_c})$, with λ as given in Table 425.4.3.2;
- c. In joist construction, for 12 mm ϕ bar and MD130 wire and smaller, a standard hook.

425.7.1.4 Anchorage of each leg of welded plain wire reinforcement forming a single U-stirrup shall be in accordance with (a) or (b):

- a. Two longitudinal wires spaced at a 50 mm spacing along the member at the top of the U;
- b. One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 50 mm from the first wire. The second wire shall be permitted to be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend of at least $8d_b$.

425.7.1.5 Anchorage of each end of a single leg stirrup of welded wire reinforcement shall be with two longitudinal wires at a minimum spacing of 50 mm with (a) and (b):

- a. Inner wire at least the greater of $d/4$ or 50 mm from $d/2$;
- b. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

425.7.1.6 Stirrups used for torsion or integrity reinforcement shall be closed stirrups perpendicular to the axis of the member. Where welded wire reinforcement is used, transverse wires shall be perpendicular to the axis of the member. Such stirrups shall be anchored by (a) or (b):

- a. Ends shall terminate with 135-degree standard hooks around a longitudinal bar;

- b. In accordance with Section 425.7.1.3(a) or (b) or Section 425.7.1.4, where the concrete surrounding the anchorage is restrained against spalling by a flange or slab or similar member.

425.7.1.6.1 Stirrups used for torsion or integrity reinforcement shall be permitted to be made up of two pieces of reinforcement: a single-U stirrup anchored according to Section 425.7.1.6(a) closed by a crosstie where the 90-degree hook of the crosstie shall be restrained against spalling by a flange or slab or similar member.

425.7.1.7 Except where used for torsion or integrity reinforcement, closed stirrups are permitted to be made using pairs of U-stirrups spliced to form a closed unit where lap lengths are at least $1.3\ell_d$. In members with a total depth of at least 450 mm, such splices with $A_b f_{yt} \leq 40$ kN per leg shall be considered adequate if stirrup legs extend the full available depth of member.

425.7.2 Ties

425.7.2.1 Ties shall consist of a closed loop of deformed bar or deformed wire with spacing in accordance with (a) and (b):

- a. Clear spacing of at least $(4/3)d_{agg}$;
- b. Center-to-center spacing shall not exceed the least of $16d_b$ of longitudinal bar, $48d_b$ of tie bar, and smallest dimension of member.

425.7.2.2 Diameter of tie bar or wire shall be at least (a) or (b):

- a. 10 mm ϕ enclosing 32 mm ϕ or smaller longitudinal bars;
- b. 12 mm ϕ enclosing 36 mm ϕ or larger longitudinal bars or bundled longitudinal bars.

425.7.2.2.1 As an alternative to deformed bars, deformed wire or welded wire reinforcement of equivalent area to that required in Section 425.7.2.1 shall be permitted subject to the requirements of Table 420.2.2.4a.

425.7.2.3 Rectilinear ties shall be arranged to satisfy (a) and (b):

- a. Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees;

- b. No unsupported bar shall be farther than 150 mm clear on each side along the tie from a laterally supported bar.

425.7.2.3.1 Anchorage of rectilinear ties shall be provided by standard hooks that conform to Section 425.3.2 and engage a longitudinal bar. A tie shall not be made up of interlocking headed deformed bars.

425.7.2.4 Circular ties shall be permitted where longitudinal bars are located around the perimeter of a circle.

425.7.2.4.1 Anchorage of individual circular ties shall be in accordance with (a) through (c):

- Ends shall overlap by at least 150 mm;
- Ends shall terminate with standard hooks in accordance with Section 425.3.2 that engage a longitudinal bar;
- Overlaps at ends of adjacent circular ties shall be staggered around the perimeter enclosing the longitudinal bars.

425.7.2.5 Ties to resist torsion shall be perpendicular to the axis of the member anchored by either (a) or (b):

- Ends shall terminate with 135-degree standard hooks or seismic hooks around a longitudinal bar;
- In accordance with Section 425.7.1.3(a) or (b) or 425.7.1.4, where the concrete surrounding the anchorage is restrained against spalling.

425.7.3 Spirals

425.7.3.1 Spirals shall consist of evenly spaced continuous bar or wire with clear spacing conforming to (a) and (b):

- At least the greater of 25 mm and $(4/3)d_{agg}$;
- Not greater than 75 mm.

425.7.3.2 For cast-in-place construction, spiral bar or wire diameter shall be at least 10 mm.

425.7.3.3 Volumetric spiral reinforcement ratio ρ_s shall satisfy Eq. 425.7.3.3.

$$\rho_s \geq 0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}} \quad (425.7.3.3)$$

where the value of f_{yt} shall not be taken greater than 700 MPa.

425.7.3.4 Spirals shall be anchored by 1-1/2 extra turns of spiral bar or wire at each end.

425.7.3.5 Spirals are permitted to be spliced by (a) or (b):

- Mechanical or welded splices in accordance with Section 425.5.7;
- Lap splices in accordance with Section 425.7.3.6 for f_{yt} not exceeding 420 MPa.

425.7.3.6 Spiral lap splices shall be at least the greater of 300 mm and the lap length in Table 425.7.3.6.

Table 425.7.3.6
Lap Length for Spiral Reinforcement

Reinforcement	Coating	Ends of lapped spiral bar or wire	Lap length mm.
Deformed bar	Uncoated or zinc-coated (galvanized)	Hook not required	$48d_b$
		Hook not required	$72d_b$
	Epoxy-coated or zinc and epoxy dual coated	Standard hook of Section 425.3.2 ^[1]	$48d_b$
Deformed wire	Uncoated	Hook not required	$48d_b$
		Hook not required	$72d_b$
	Epoxy-coated	Standard hook of Section 425.3.2 ^[1]	$48d_b$
Plain bar	Uncoated or zinc-coated (galvanized)	Hook not required	$72d_b$
		Standard hook of Section 425.3.2 ^[1]	$48d_b$
Plain wire	Uncoated	Hook not required	$72d_b$
		Standard hook of Section 425.3.2 ^[1]	$48d_b$

^[1] Hooks shall be embedded within the core confined by the spiral.

425.7.4 Hoops

425.7.4.1 Hoops shall consist of a closed tie or continuously wound tie, which can consist of several reinforcement elements each having seismic hooks at both ends.

425.7.4.2 Anchorage of the ends of the reinforcement elements that comprise hoops shall be provided by seismic hooks that conform to Section 425.3.4 and engage a longitudinal bar. A closed tie shall not be made up of interlocking headed deformed bars.

425.8 Post-Tensioning Anchorages and Couplers

425.8.1 Anchorages and couplers for tendons shall develop at least 95 percent of f_{pu} when tested in an unbonded condition, without exceeding anticipated set.

425.8.2 Anchorages and couplers for bonded tendons shall be located so that 100 percent of f_{pu} shall be developed at critical sections after the post-tensioned reinforcement is bonded in the member.

425.8.3 In unbonded construction subject to repetitive loads, the possibility of fatigue of prestressed reinforcement in anchorages and couplers shall be considered.

425.8.4 Couplers shall be placed at locations approved by the licensed design professional and enclosed in housings long enough to permit necessary movements.

425.9 Anchorage Zones for Post-Tensioned Tendons

425.9.1 General

425.9.1.1 Anchorage regions of post-tensioned tendons shall consist of two zones, (a) and (b):

- a. The local zone shall be assumed to be a rectangular prism (or equivalent rectangular prism for circular or oval anchorages) of concrete immediately surrounding the anchorage device and any confining reinforcement;
- b. The general zone includes the local zone and shall be assumed to be the portion of the member through which the concentrated prestressing force is transferred to the concrete and distributed more uniformly across the section.

425.9.1.2 The local zone shall be designed in accordance with Section 425.9.3.

425.9.1.3 The general zone shall be designed in accordance with Section 425.9.4.

425.9.1.4 Compressive strength of concrete required at time of post-tensioning shall be specified as required by Section 426.10

425.9.1.5 Stressing sequence shall be considered in the design process and specified as required by Section 426.10.

425.9.2 Required Strength

425.9.2.1 Factored prestressing force at the anchorage device shall exceed the least of (a) through (c), where 1.2 is the load factor from Section 405.3.12:

- a. $1.2(0.94f_{py})A_{ps}$;
- b. $1.2(0.80f_{pu})A_{ps}$;
- c. Maximum jacking force designated by the supplier of anchorage devices multiplied by 1.2.

425.9.3 Local Zone

425.9.3.1 The design of local zone in post-tensioned anchorages shall meet the requirements of (a), (b), or (c):

- a. Mono strand or single 16 mm or smaller diameter bar anchorage devices shall meet the bearing resistance and local zone requirements of ACI 423.7;
- b. Basic multi-strand anchorage devices shall meet the bearing resistance requirements of AASHTO LRFD Bridge Design Specifications, Article 5.10.9.7.2, except that the load factors shall be in accordance with Section 405.3.12 and ϕ shall be in accordance with Section 421.2.1;
- c. Special anchorage devices shall satisfy the tests required in AASHTO LRFD Bridge Design Specifications, Article 5.10.9.7.3, and described in AASHTO LRFD Bridge Construction Specifications, Article 10.3.2.3.

425.9.3.2 Where special anchorage devices are used, supplementary skin reinforcement shall be provided in addition to the confining reinforcement specified for the anchorage device.

425.9.3.2.1 Supplementary skin reinforcement shall be similar in configuration and at least equivalent in volumetric ratio to any supplementary skin reinforcement used in the qualifying acceptance tests of the anchorage device.

425.9.4 General Zone

425.9.4.1 The extent of the general zone is equal to the largest dimension of the cross section. In the case of slabs with anchorages or groups of anchorages spaced along the

slab edge, the depth of the general zone shall be taken as the spacing of the tendons.

425.9.4.2 For anchorage devices located away from the end of a member, the general zone shall include the disturbed regions ahead of and behind the anchorage devices.

425.9.4.3 Analysis of General Zones

425.9.4.3.1 Methods (a) through (c) shall be permitted for design of general zones:

- a. Strut-and-tie models in accordance with Section 423;
- b. Linear stress analysis, including finite element analysis or equivalent;
- c. Simplified equations in AASHTO LRFD Bridge Design Specifications, Article 5.10.9.6, except where restricted by Section 425.9.4.3.2.

The design of general zones by other methods shall be permitted, provided that the specific procedures used for design result in prediction of strength in substantial agreement with results of comprehensive tests.

425.9.4.3.2 Simplified equations as permitted by Section 425.9.4.3.1(c) shall not be used where (a), (b), (c), (d), (e), (f), or (g) occur:

- a. Member cross sections are non-rectangular;
- b. Discontinuities in or near the general zone cause deviations in the force flow path;
- c. Minimum edge distance is less than 1.5 times the anchorage device lateral dimension in that direction;
- d. Multiple anchorage devices are used in other than one closely spaced group;
- e. Centroid of the tendons is located outside the kern;
- f. Angle of inclination of the tendon in the general zone is less than -5 degrees from the centerline of axis of the member, where the angle is negative if the anchor force points away from the centroid of the section;
- g. Angle of inclination of the tendon in the general zone is greater than +20 degrees from the centerline of axis of the member, where the angle is positive if the anchor force points towards the centroid of the section.

425.9.4.3.3 Three-dimensional effects shall be considered in design and analyzed by (a) or (b):

- a. Three-dimensional analysis procedures;
- b. Approximated by considering the summation of effects for two orthogonal planes.

425.9.4.4 Reinforcement Limits

425.9.4.4.1 Tensile strength of concrete shall be neglected in calculations of reinforcement requirements.

425.9.4.4.2 Reinforcement shall be provided in the general zone to resist bursting, spalling, and longitudinal edge tension forces induced by anchorage devices, as applicable. Effects of abrupt changes in section and stressing sequence shall be considered.

425.9.4.4.3 For anchorage devices located away from the end of the member, bonded reinforcement shall be provided to transfer at least $0.35P_{pu}$ into the concrete section behind the anchor. Such reinforcement shall be placed symmetrically around the anchorage device and shall be fully developed both behind and ahead of the anchorage device.

425.9.4.4.4 If tendons are curved in the general zone, bonded reinforcement shall be provided to resist radial and splitting forces, except for monostrand tendons in slabs or where analysis shows reinforcement is not required.

425.9.4.4.5 Reinforcement with a nominal tensile strength equal to 2 percent of the factored prestressing force shall be provided in orthogonal directions parallel to the loaded face of the anchorage zone to limit spalling, except for monostrand tendons in slabs or where analysis shows reinforcement is not required.

425.9.4.4.6 For monostrand anchorage devices for 12 mm. or smaller diameter strands in normal-weight concrete slabs, reinforcement satisfying (a) and (b) shall be provided in the general zone, unless a detailed analysis in accordance with Section 425.9.4.3 shows that this reinforcement is not required:

- a. Two horizontal bars at least 12 mm ϕ in size shall be provided parallel to the slab edge. They shall be permitted to be in contact with the front face of the anchorage device and shall be within a distance of ahead of each device. Those bars shall extend at least 150 mm. either side of the outer edges of each device;

- b. If the center-to-center spacing of anchorage devices is 300 mm or less, the anchorage devices shall be considered as a group. For each group of six or more anchorage devices, $n + 1$ hairpin bars or closed stirrups at least 10 mm ϕ in size shall be provided, where n is the number of anchorage devices. One hairpin bar or stirrup shall be placed between each anchorage device and one on each side of the group. The hairpin bars or stirrups shall be placed with the legs extending into the slab perpendicular to the edge. The center portion of the hairpin bars or stirrups shall be placed perpendicular to the plane of the slab from $3h/8$ to $h/2$ ahead of the anchorage devices.

- b. Prestressing reinforcement is stressed to no more than 50 percent of the final prestressing force.

425.9.5 Reinforcement Detailing

425.9.5.1 Selection of reinforcement size, spacing, cover, and other details for anchorage zones shall make allowances for tolerances on fabrication and placement of reinforcement; for the size of aggregate; and for adequate placement and consolidation of the concrete.

425.9.4.5 Limiting Stresses in General Zones

425.9.4.5.1 Maximum design tensile stress in reinforcement at nominal strength shall not exceed the limits in Table 425.9.4.5.1.

Table 425.9.4.5.1
Maximum Design Tensile Stress in Reinforcement

Type of reinforcement	Maximum design tensile stress
Non-prestressed reinforcement	f_y
Bonded, prestressed reinforcement	f_{py}
Unbonded, prestressed reinforcement	$f_{se} + 70$

425.9.4.5.2 Compressive stress in concrete at nominal strength shall not exceed $0.7\lambda f'_{ci}$, where λ is defined in Section 419.2.4.

425.9.4.5.3 If concrete is confined by spirals or hoops and the effect of confining reinforcement is documented by tests and analysis, it shall be permitted to use an increased value of compressive stress in concrete when calculating the nominal strength of the general zone.

425.9.4.5.4 Prestressing reinforcement shall not be stressed until compressive strength of concrete, as indicated by tests of cylinders cured in a manner consistent with curing of the member, is at least 17 MPa for single-strand or bar tendons or at least 28 MPa for multistrand tendons unless Section 425.9.4.5.5 is satisfied.

425.9.4.5.5 Provisions of Section 425.9.4.5.4 need not be satisfied if (a) or (b) is satisfied:

- a. Oversized anchorage devices are used to compensate for a lower concrete compressive strength;

SECTION 426

CONSTRUCTION DOCUMENTS AND INSPECTION

426.1 Scope

426.1.1 This section addresses (a) through (c):

- Design information that the licensed design professional shall specify in the construction documents, if applicable;
- Compliance requirements that the licensed design professional shall specify in the construction documents, if applicable;
- Inspection requirements that the licensed design professional shall specify in the construction documents, if applicable.

426.2 Design Criteria

426.2.1 Design Information:

- Name and year of issue of the Code, general building code, and any supplements governing design.
- Loads used in design.
- Design work delegated to the contractor including applicable design criteria.

426.3 Member Information

426.3.1 Design Information:

- Member size, location, and related tolerances.

426.4 Concrete Materials and Mixture Requirements

426.4.1 Concrete Materials

426.4.1.1 Cementitious Materials

426.4.1.1.1 Compliance Requirements:

- Cementitious materials shall conform to the specifications in Table 426.4.1.1.1 (a);

Table 426.4.1.1.1 (a)
Specifications for Cementitious Materials

Cementitious Material	Specification
Portland cement	ASTM C150M
Blended hydraulic cements	ASTM C595M, excluding Type IS (≥ 70) and Type IT ($S \geq 70$)
Expansive hydraulic cement	ASTM C845M
Hydraulic cement	ASTM C1157M
Fly ash and natural pozzolan	ASTM C618
Slag cement	ASTM C989M
Silica fume	ASTM C1240

- All cementitious materials specified in Table 426.4.1.1.1(a) and the combinations of these materials shall be included in calculating the w/cm of the concrete mixture.

426.4.1.2 Aggregates

426.4.1.2.1 Compliance Requirements:

- Aggregates shall conform to (1) or (2):
 - Normal weight aggregate: ASTM C33M;
 - Lightweight aggregate: ASTM C330M.
- Aggregates not conforming to ASTM C33M or ASTM C330M are permitted if they have been shown by test or actual service to produce concrete of adequate strength and durability and are approved by the building official.

426.4.1.3 Water

426.4.1.3.1 Compliance Requirements:

- Mixing water shall conform to ASTM C1602M;
- Mixing water, including that portion of mixing water contributed in the form of free moisture on aggregates, shall not contain deleterious amounts of chloride ion when used for prestressed concrete, for concrete that will contain aluminum embedments, or for concrete cast against stay-in-place galvanized steel forms.

426.4.1.4 Admixtures

- a. Admixtures shall conform to (1) through (4):
1. Water reduction and setting time modification: ASTM C494M;
 2. Producing flowing concrete: ASTM C1017M;
 3. Air entrainment: ASTM C260M;
 4. Inhibiting chloride-induced corrosion: ASTM C1582M.
- b. Admixtures that do not conform to the specifications in Section 426.4.1.4.1(a) shall be subject to prior review by the licensed design professional;
- c. Calcium chloride or admixtures containing chloride from sources other than impurities in admixture ingredients shall not be used in prestressed concrete, in concrete containing embedded aluminum, or in concrete cast against stay-in-place galvanized steel forms;
- d. Admixtures used in concrete containing expansive cements conforming to ASTM C845M shall be compatible with the cement and produce no deleterious effects.

426.4.1.5 Steel Fiber Reinforcement**426.4.1.5.1 Compliance Requirements:**

- a. Steel fiber reinforcement used for shear resistance shall satisfy (1) and (2):
1. Be deformed and conform to ASTM A820M;
 2. Have a length-to-diameter ratio of at least 50 and not exceeding 100.

426.4.2 Concrete Mixture Requirements**426.4.2.1 Design Information:**

- a. Requirements (1) through (11) for each concrete mixture, based on assigned exposure classes or design of members:
1. Minimum specified compressive strength of concrete, f'_c ;
 2. Test age for demonstrating compliance with f'_c if different from 28 days;

3. Maximum w/cm applicable to most restrictive assigned durability exposure class from Section 419.3.2.1;
4. Nominal maximum size of coarse aggregate not to exceed the least of (a), (b), and (c):
 - a. 1/5 the narrowest dimension between sides of forms;
 - b. 1/3 the depth of slabs;
 - c. 3/4 the minimum specified clear spacing between individual reinforcing bars or wires, bundles of bars, prestressed reinforcement, individual tendons, bundled tendons, or ducts.

These limitations shall not apply if, in the judgment of the licensed design professional, workability and methods of consolidation are such that concrete can be placed without honeycombs or voids.

5. For members assigned to Exposure Category F, air content from Section 419.3.3.1;
 6. For members assigned to Exposure Class C, applicable chloride ion limits for assigned Exposure Class from 4 Section 419.3.2.1;
 7. For members assigned to Exposure Category S, type of cementitious materials for assigned Exposure Class from 419.3.2.1;
 8. For members assigned to Exposure Class S2 or S3, admixtures containing calcium chloride are prohibited;
 9. Equilibrium density of lightweight concrete;
 10. Requirement for submittal of the volumetric fractions of aggregate in lightweight concrete mixtures for the verification of λ value if used in design;
 11. If used for shear resistance in accordance with Section 409.6.3.1, requirements for steel fiber reinforced concrete.
- b. At the option of the licensed design professional, exposure classes based on the severity of the anticipated exposure of members;
- c. The required compressive strength at designated stages of construction for each part of the structure designed by the licensed design professional.

426.4.2.2 Compliance Requirements:

- a. The required compressive strength at designated stages of construction for each part of the structure not designed by the licensed design professional shall be submitted for review;
- b. The maximum percentage of pozzolans, including fly ash and silica fume, and slag cement in concrete assigned to Exposure Class F3, shall be in accordance with Table 426.4.2.2(b) and (1) and (2).
 1. The maximum percentage limits in Table 426.4.2.2(b) shall include the fly ash or other pozzolans, slag cement, and silica fume used in the manufacture of ASTM C595M and C1157M blended cements;
 2. The individual limits in Table 426.4.2.2(b) shall apply regardless of the number of cementitious materials in a concrete mixture.

Table 426.4.2.2(b)
Limits on Cementitious Materials for Concrete
Assigned to Exposure Class F3

Cementitious Materials	Maximum percent of total cementitious materials by mass
Fly ash or other pozzolans conforming to ASTM C618	25
Slag cement conforming to ASTM C989M	50
Silica fume conforming to ASTM C1240	10
Total of fly ash or other pozzolans and silica fume	35
Total of fly ash or other pozzolans, slag cement, and silica fume	50

Table 426.4.2.2(c)
Requirements for Establishing Suitability of Combinations of
Cementitious Materials Exposed to Water-Soluble Sulfate

Exposure class	Maximum expansion strain if tested using ASTM C1012M		
	At 6 months	At 12 months	At 18 months
S1	0.10 percent	No requirement	No requirement
S2	0.05 percent	0.10 percent ^[1]	No requirement
S3	No requirement	No requirement	0.10 percent

^[1] The 12-month expansion limit applies only if the measured expansion exceeds the 6-month maximum expansion limit.

- c. For concrete exposed to sulfate, alternative combinations of cementitious materials to those specified in Section 426.4.2.1(a)(7) are permitted if tests for sulfate resistance satisfy the criteria in Table 426.4.2.2(c).
 1. Conform to ASTM C1116M;
 2. Contain at least 60 kg of deformed steel fibers per cubic meter of concrete.
- d. Steel fiber-reinforced concrete used for shear resistance shall satisfy (1) and (2):
 1. Can be placed readily without segregation into forms and around reinforcement under anticipated placement conditions;
 2. Meets requirements for assigned exposure class in accordance with either Section 426.4.2.1(a) or 426.4.2.1(b);
 3. Conforms to strength test requirements for standard-cured specimens.
- b. Concrete mixture proportions shall be established in accordance with Article 4.2.3 of ACI 301M or by an alternative method acceptable to the licensed design professional. Alternative methods shall have a probability of satisfying the strength requirements for acceptance tests of standard-cured specimens that meets or exceeds the probability associated with the method in Article 4.2.3 of ACI 301M. If Article 4.2.3 of ACI 301M is used, the strength test records used for establishing and documenting concrete mixture proportions shall not be more than 24 months old;

426.4.3 Proportioning of Concrete Mixtures**426.4.3.1 Compliance Requirements:**

- a. Concrete mixture proportions shall be established so that the concrete satisfies (1) through (3):

- c. The concrete materials used to develop the concrete mixture proportions shall correspond to those to be used in the proposed Work;
- d. If different concrete mixtures are to be used for different portions of proposed Work, each mixture shall comply with the concrete mixture requirements stated in the construction documents.

426.4.4 Documentation of Concrete Mixture Characteristics

426.4.4.1 Compliance Requirements:

- a. Documentation of concrete mixture characteristics shall be submitted for review by the licensed design professional before the mixture is used and before making changes to mixtures already in use. Evidence of the ability of the proposed mixture to comply with the concrete mixture requirements in the construction documents shall be included in the documentation. The evidence shall be based on field test records or laboratory trial batches. Field test records shall represent conditions similar to those anticipated during the proposed Work;
- b. If field or laboratory test data are not available, and $f'_c \leq 35$ MPa, concrete proportions shall be based on other experience or information, if approved by the licensed design professional. If $f'_c > 35$ MPa, test data documenting the characteristics of the proposed mixtures are required;
- c. If data become available during construction that consistently exceed the strength-test acceptance criteria for standard-cured specimens, it shall be permitted to modify mixture to reduce the average strength. Submit evidence acceptable to the licensed design professional to demonstrate that the modified mixture will comply with the concrete mixture requirements in the construction documents.

426.5 Concrete Production and Construction

426.5.1 Concrete Production

426.5.1.1 Compliance Requirements:

- a. Cementitious materials and aggregates shall be stored to prevent deterioration or contamination;
- b. Material that has deteriorated or has been contaminated shall not be used in concrete;
- c. Equipment for mixing and transporting concrete shall conform to ASTM C94M or ASTM C685M;

- d. Ready-mixed and site-mixed concrete shall be batched, mixed, and delivered in accordance with ASTM C94M or ASTM C685M.

426.5.2 Concrete Placement and Consolidation

426.5.2.1 Compliance Requirements:

- a. Debris and ice shall be removed from spaces to be occupied by concrete before placement;
- b. Standing water shall be removed from place of deposit before concrete is placed unless a tremie is to be used or unless otherwise permitted by both the licensed design professional and the building official;
- c. Masonry filler units that will be in contact with concrete shall be pre-wetted prior to placing concrete;
- d. Equipment used to convey concrete from the mixer to the location of final placement shall have capabilities to achieve the placement requirements;
- e. Concrete shall not be pumped through pipe made of aluminum or aluminum alloys;
- f. Concrete shall be placed in accordance with (1) through (5):
 1. At a rate to provide an adequate supply of concrete at the location of placement;
 2. At a rate so concrete at all times has sufficient workability such that it can be consolidated by the intended methods;
 3. Without segregation or loss of materials;
 4. Without interruptions sufficient to permit loss of workability between successive placements that would result in cold joints;
 5. Deposited as near to its final location as practicable to avoid segregation due to rehandling or flowing.
- g. Concrete that has been contaminated or has lost its initial workability to the extent that it can no longer be consolidated by the intended methods shall not be used;
- h. Retempering concrete in accordance with the limits of ASTM C94M shall be permitted unless otherwise restricted by the licensed design professional;

- i. After starting, concreting shall be carried on as a continuous operation until the completion of a panel or section, as defined by its boundaries or predetermined joints;
- j. Concrete shall be consolidated by suitable means during placement and shall be worked around reinforcement and embedments and into corners of forms;
- k. Top surfaces of vertically formed lifts shall be generally level.

426.5.3 Curing Concrete

426.5.3.1 Design Information:

- a. If supplementary tests of field-cured specimens are required to verify adequacy of curing and protection, the number and size of test specimens and the frequency of these supplementary tests.

426.5.3.2 Compliance Requirements:

- a. Concrete, other than high-early-strength, shall be maintained at a temperature of at least 10°C and in a moist condition for at least the first 7 days after placement, except if accelerated curing is used;
- b. High-early-strength concrete shall be maintained at a temperature of at least 10°C and in a moist condition for at least the first 3 days after placement, except if accelerated curing is used;
- c. Accelerated curing to accelerate strength gain and reduce time of curing is permitted using high pressure steam, steam at atmospheric pressure, heat and moisture, or other process acceptable to the licensed design professional. If accelerated curing is used, (1) and (2) shall apply:
 - 1. Compressive strength at the load stage considered shall be at least the strength required at that load stage.
 - 2. Accelerated curing shall not impair the durability of the concrete.
- d. If required by the building official or licensed design professional, results of tests of cylinders made and cured in accordance with (1) and (2) shall be provided in addition to results of standard-cured cylinder strength tests:
 - 1. At least two 150 by 300 mm or at least three 100 by 200 mm field-cured cylinders shall be molded

at the same time and from the same samples as standard-cured cylinders;

- 2. Field-cured cylinders shall be cured in accordance with the field curing procedure of ASTM C31M and tested in accordance with ASTM C39M.
- e. Procedures for protecting and curing concrete shall be considered adequate if (1) or (2) are satisfied:
 - 1. Average strength of field-cured cylinders at test age designated for determination of f'_c is equal to or at least 85 percent of that of companion standard-cured cylinders;
 - 2. Average strength of field-cured cylinders at test age exceeds f'_c by more than 3.5 MPa.

426.5.4 Concreting in Cold Weather

426.5.4.1 Design Information:

426.5.4.1 (a) Temperature limits for concrete as delivered in cold weather.

426.5.4.2 Compliance Requirements:

- a. Adequate equipment shall be provided for heating concrete materials and protecting concrete during freezing or near-freezing weather;
- b. Frozen materials or materials containing ice shall not be used;
- c. Forms, fillers, and ground with which concrete is to come in contact shall be free from frost and ice;
- d. Concrete materials and production methods shall be selected so that the concrete temperature at delivery complies with the specified temperature limits.

426.5.5 Concreting in Hot Weather

426.5.5.1 Design Information:

426.5.5.1 (a) Temperature limits for concrete as delivered in hot weather.

426.5.5.2 Compliance Requirements:

- a. Concrete materials and production methods shall be selected so that the concrete temperature at delivery complies with the specified temperature limits;

- b. Handling, placing, protection, and curing procedures shall limit concrete temperatures or water evaporation that could reduce strength, serviceability, and durability of the member or structure.

426.5.6 Construction, Contraction, and Isolation Joints

426.5.6.1 Design Information:

- a. If required by the design, locations and details of construction, isolation, and contraction joints;
- b. Details required for transfer of shear and other forces through construction joints;
- c. Surface preparation, including intentional roughening of hardened concrete surfaces where concrete is to be placed against previously hardened concrete;
- d. Locations where shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars requiring steel to be clean and free of paint;
- e. Surface preparation including intentional roughening if composite topping slabs are to be cast in place on a precast floor or roof intended to act structurally with the precast members.

426.5.6.2 Compliance Requirements:

- a. Joint locations or joint details not shown or that differ from those indicated in construction documents shall be submitted for review by the licensed design professional;
- b. Except for prestressed concrete, construction joints in floor and roof systems shall be located within the middle third of spans of slabs, beams, and girders unless otherwise approved by the licensed design professional;
- c. Construction joints in girders shall be offset a distance of at least two times the width of intersecting beams, measured from the face of the intersecting beam, unless otherwise approved by the licensed design professional;
- d. Surface of concrete construction joints shall be intentionally roughened if specified. Otherwise, construction joints shall be cleaned and laitance removed before new concrete is placed;

- e. Immediately before new concrete is placed, construction joints shall be pre-wetted and standing water removed.

426.5.7 Construction of Concrete Members

426.5.7.1 Design Information:

- a. Details required to accommodate dimensional changes resulting from prestressing, creep, shrinkage, and temperature;
- b. Identify if a slab-on-ground is designed as a structural diaphragm or part of the seismic-force-resisting system;
- c. Details for construction of sloped or stepped footings designed to act as a unit;
- d. Locations where slab and column concrete placements are required to be integrated during placement in accordance with Section 415.3;
- e. Locations where steel fiber-reinforced concrete is required for shear resistance in accordance with Section 409.6.3.1.

426.5.7.2 Compliance Requirements:

- a. Beams, girders, or slabs supported by columns or walls shall not be cast until concrete in the vertical support members is no longer plastic;
- b. Beams, girders, haunches, drop panels, shear caps, and capitals shall be placed monolithically as part of a slab system, unless otherwise shown in construction documents;
- c. At locations where slab and column concrete placements are required to be integrated during placement, column concrete shall extend full slab depth at least 0.60 m into floor slab from face of column and be integrated with floor concrete;
- d. Saw-cutting in slabs-on-ground identified in the construction documents as structural diaphragms or part of the seismic-force-resisting system shall not be permitted unless specifically indicated or approved by the licensed design professional.

426.6 Reinforcement Materials and Construction Requirements

426.6.1 General

426.6.1.1 Design Information

- ASTM designation and grade of reinforcement;
- Type, size, location requirements, detailing, and embedment length of reinforcement;
- Concrete cover to reinforcement;
- Location and length of lap splices;
- Type and location of mechanical splices;
- Type and location of end-bearing splices;
- Type and location of welded splices and other required welding of reinforcing bars;
- ASTM designation for protective coatings of non-prestressed reinforcement;
- Corrosion protection for exposed reinforcement intended to be bonded with extensions on future Work.

426.6.1.2 Compliance Requirements

- Mill test reports for reinforcement shall be submitted;
- Non-prestressed reinforcement with rust, mill scale, or a combination of both shall be considered satisfactory, provided a hand-wire-brushed representative test specimen of the reinforcement complies with the applicable ASTM specification for the minimum dimensions (including height of deformations) and weight per unit length;
- Prestressing reinforcement shall be free of mill scale, pitting, and excessive rust. A light coating of rust shall be permitted;
- At the time concrete is placed, reinforcement to be bonded shall be clean of mud, oil, or other deleterious coatings that decrease bond.

426.6.2 Placement

426.6.2.1 Design Information.

- Tolerances on location of reinforcement taking into consideration tolerances on d and specified concrete cover in accordance with Table 426.6.2.1(a);

Table 426.6.2.1(a)
Tolerances on d and Specified Cover

d , mm.	Tolerance on d , mm.	Tolerance on specified concrete cover, mm. ^[1]	
≤ 200	± 10	Smaller of	-10
			$-\left(\frac{1}{3}\right) * \text{specified cover}$
> 200	± 12	Smaller of	-12
			$-\left(\frac{1}{3}\right) * \text{specified cover}$

^[1] Tolerance for cover to formed soffits is -6 mm.

- Tolerance for longitudinal location of bends and ends of reinforcement in accordance with Table 426.6.2.1(b). The tolerance for specified concrete cover in Table 426.6.2.1(a) shall also apply at discontinuous ends of members.

Table 426.6.2.1(b)
Tolerances for Longitudinal Location of Bends and Ends of Reinforcement

Location of bends or reinforcement ends	Tolerances, mm.
Discontinuous ends of brackets and corbels	± 12
Discontinuous ends of other members	± 25
Other locations	± 50

426.6.2.2 Compliance Requirements

- Reinforcement, including bundled bars, shall be placed within required tolerances and supported to prevent displacement beyond required tolerances during concrete placement;
- Spiral units shall be continuous bar or wire placed with even spacing and without distortion beyond the tolerances for the specified dimensions;
- Splices of reinforcement shall be made only as permitted in the construction documents, or as authorized by the licensed design professional;
- For longitudinal column bars forming an endbearing splice, the bearing of square cut ends shall be held in concentric contact;

- e. Bar ends shall terminate in flat surfaces within 1.5 degrees of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly.

426.6.3 Bending

426.6.3.1 Compliance Requirements

- a. Reinforcement shall be bent cold prior to placement, unless otherwise permitted by the licensed design professional;
- b. Field bending of reinforcement partially embedded in concrete shall not be permitted, except as shown in the construction documents or permitted by the licensed design professional;
- c. Offset bars shall be bent before placement in the forms.

426.6.4 Welding

426.6.4.1 Compliance Requirements.

- a. Welding of all non-prestressed bars shall conform to the requirements of AWS D1.4M. ASTM specifications for bar reinforcement, except for ASTM A706M, shall be supplemented to require a mill test report of material properties that demonstrate conformance to the requirements in AWS D1.4M;
- b. Welding of crossing bars shall not be used for assembly of reinforcement unless permitted by the licensed design professional.

426.7 Anchoring to Concrete

426.7.1 Design Information

- a. Requirements for assessment and qualification of anchors for the applicable conditions of use in accordance with Section 417.1.3;
- b. Type, size, location requirements, effective embedment depth, and installation requirements for anchors;
- c. Minimum edge distance of anchors in accordance with Section 417.7;
- d. Inspection requirements in accordance with Section 426.13;

- e. For post-installed anchors, parameters associated with the strength used for design, including anchor category, concrete strength, and aggregate type;

- f. For adhesive anchors, parameters associated with the characteristic bond stress used for design in accordance with Section 417.4.5 including minimum age of concrete, concrete temperature range, moisture condition of concrete at time of installation, type of lightweight concrete if applicable, and requirements for hole drilling and preparation;

- g. Qualification requirements for installers of anchors in accordance with Section 417.8.1;

- h. Adhesive anchors installed in a horizontal or upwardly inclined orientation, if they support sustained tension loads;

- i. Required certifications for installers of adhesive anchors that are installed in a horizontal or upwardly inclined orientation to support sustained tension loads in accordance with Sections 417.8.2.2 and 417.8.2.3;

- j. For adhesive anchors, proof loading where required in accordance with Section 417.8.2.1;

- k. Corrosion protection for exposed anchors intended for attachment with future Work.

426.7.2 Compliance Requirements

- a. Post-installed anchors shall be installed in accordance with the Manufacturer's Printed Installation Instructions (MPII).

426.8 Embedments

426.8.1 Design Information:

- a. Type, size, details, and location of embedments designed by the licensed design professional;
- b. Reinforcement required to be placed perpendicular to pipe embedments;
- c. Specified concrete cover for pipe embedments with their fittings;
- d. Corrosion protection for exposed embedments intended to be connected with future Work.

426.8.2 Compliance Requirements:

- a. Type, size, details, and location of embedments not shown in the construction documents shall be

submitted for review by the licensed design professional;

- b. Aluminum embedments shall be coated or covered to prevent aluminum-concrete reaction and electrolytic action between aluminum and steel;
- c. Pipes and fittings not shown in the construction documents shall be designed to resist effects of the material, pressure, and temperature to which they will be subjected;
- d. No liquid, gas, or vapor, except water not exceeding 32 °C or 0.35 MPa pressure, shall be placed in the pipes until the concrete has attained its specified strength;
- e. In solid slabs, piping, except for radiant heating or snow melting, shall be placed between top and bottom reinforcement;
- f. Conduit and piping shall be fabricated and installed so that cutting, bending, or displacement of reinforcement from its specified location is not required.

426.9 Additional Requirements for Precast Concrete

426.9.1 Design Information:

- a. Dimensional tolerances for precast members and interfacing members;
- b. Details of lifting devices, embedments, and related reinforcement required to resist temporary loads from handling, storage, transportation, and erection, if designed by the licensed design professional.

426.9.2 Compliance Requirements:

- a. Members shall be marked to indicate location and orientation in the structure and date of manufacture;
- b. Identification marks on members shall correspond to erection drawings;
- c. Design and details of lifting devices, embedments, and related reinforcement required to resist temporary loads from handling, storage, transportation, and erection shall be provided if not designed by the licensed design professional;
- d. During erection, precast members and structures shall be supported and braced to ensure proper alignment,

strength, and stability until permanent connections are completed;

- e. If approved by the licensed design professional, items embedded while the concrete is in a plastic state shall satisfy (1) through (4):
 - 1. Embedded items shall protrude from the precast concrete members or remain exposed for inspection;
 - 2. Embedded items are not required to be hooked or tied to reinforcement within the concrete;
 - 3. Embedded items shall be maintained in the correct position while the concrete remains plastic;
 - 4. The concrete shall be consolidated around embedded items.

426.10 Additional Requirements for Prestressed Concrete

426.10.1 Design Information:

- a. Magnitude and location of prestressing forces;
- b. Stressing sequence of tendons;
- c. Type, size, details, and location of post-tensioning anchorages for systems selected by the licensed design professional;
- d. Tolerances for placement of tendons and post-tensioning ducts in accordance with Table 426.6.2.1(a);
- e. Materials and details of corrosion protection for unbonded tendons, external tendons, couplers, end fittings, post-tensioning anchorages, and anchorage regions;
- f. Requirements for ducts for bonded tendons;
- g. Requirements for grouting of bonded tendons, including maximum water-soluble chloride ion (Cl⁻) content requirements in Section 419.4.1.

426.10.2 Compliance Requirements:

- a. Type, size, details, and location of post-tensioning anchorage systems not shown in the construction documents shall be submitted to the licensed design professional for review;

- b. Tendons and post-tensioning ducts shall be placed within required tolerances and supported to prevent displacement beyond required tolerances during concrete placement;
- c. Couplers shall be placed in areas approved by the licensed design professional and enclosed in housings long enough to permit necessary movements;
- d. Burning or welding operations in the vicinity of prestressing reinforcement shall be performed in such a manner that prestressing reinforcement is not subject to welding sparks, ground currents, or temperatures that degrade the properties of the reinforcement;
- e. Prestressing force and friction losses shall be verified by (1) and (2):
 - 1. Measured elongation of prestressed reinforcement compared with elongation calculated using the modulus of elasticity determined from tests or as reported by the manufacturer;
 - 2. Jacking force measured using calibrated equipment such as a hydraulic pressure gauge, load cell, or dynamometer.
- f. The cause of any difference in force determination between (1) and (2) of Section 426.10.2(e) that exceeds 5 percent for pretensioned construction or 7 percent for post-tensioned construction shall be ascertained and corrected, unless otherwise approved by the licensed design professional;
- g. Loss of prestress force due to unreplaced broken prestressed reinforcement shall not exceed 2 percent;
- h. If the transfer of force from the anchorages of the pretensioning bed to the concrete is accomplished by flame cutting prestressed reinforcement, the cutting locations and cutting sequence shall be selected to avoid undesired temporary stresses in pretensioned members;
- i. Long lengths of exposed pretensioned strand shall be cut near the member to minimize shock to the concrete;
- j. Prestressing reinforcement in post-tensioned construction shall not be stressed until the concrete compressive strength is at least 17 MPa for single-strand or bar tendons, 28 MPa for multi-strand tendons, or a higher strength, if required. An

exception to these strength requirements is provided in Section 426.10.2(k);

- k. Lower concrete compressive strength than required by Section 426.10.2(j) shall be permitted if (1) or (2) is satisfied:
 - 1. Oversized anchorage devices are used to compensate for a lower concrete compressive strength;
 - 2. Prestressing reinforcement is stressed to no more than 50 percent of the final prestressing force.

426.11 Formwork

426.11.1 Design of Formwork

426.11.1.1 Design Information:

- a. Requirement for the contractor to design, fabricate, install, and remove formwork;
- b. Location of composite members requiring shoring;
- c. Requirements for removal of shoring of composite members.

426.11.1.2 Compliance Requirements:

- a. Design of formwork shall consider (1) through (5):
 - 1. Method of concrete placement;
 - 2. Rate of concrete placement;
 - 3. Construction loads, including vertical, horizontal, and impact;
 - 4. Avoidance of damage to previously constructed members;
 - 5. For post-tensioned members, allowance for movement of the member during application of the prestressing force without damage to the member.
- b. Formwork fabrication and installation shall result in a final structure that conforms to shapes, lines, and dimensions of the members as required by the construction documents;
- c. Formwork shall be sufficiently tight to inhibit leakage of paste or mortar;

- d. Formwork shall be braced or tied together to maintain position and shape.

426.11.2 Removal of Formwork

426.11.2.1 Compliance requirements:

- a. Before starting construction, the contractor shall develop a procedure and schedule for removal of formwork and installation of reshores, and shall calculate the loads transferred to the structure during this process;
- b. Structural analysis and concrete strength requirements used in planning and implementing the formwork removal and reshore installation shall be furnished by the contractor to the licensed design professional and, when requested, to the building official;
- c. No construction loads shall be placed on, nor any formwork removed from, any part of the structure under construction except when that portion of the structure in combination with remaining formwork has sufficient strength to support safely its weight and loads placed thereon and without impairing serviceability;
- d. Sufficient strength shall be demonstrated by structural analysis considering anticipated loads, strength of formwork, and an estimate of in-place concrete strength;
- e. The estimate of in-place concrete strength shall be based on tests of field-cured cylinders or on other procedures to evaluate concrete strength approved by the licensed design professional and, when requested, approved by the building official;
- f. Formwork shall be removed in such a manner not to impair safety and serviceability of the structure;
- g. Concrete exposed by formwork removal shall have sufficient strength not to be damaged by the removal;
- h. Formwork supports for post-tensioned members shall not be removed until sufficient post-tensioning has been applied to enable post-tensioned members to support their dead load and anticipated construction loads;
- i. No construction loads exceeding the combination of superimposed dead load plus live load including reduction shall be placed on any unshored portion of the structure under construction, unless analysis indicates adequate

strength to support such additional loads and without impairing serviceability.

426.12 Concrete Evaluation and Acceptance

426.12.1 General

426.12.1.1 Compliance Requirements:

- a. A strength test shall be the average of the strengths of at least two 150×300 mm cylinders or at least three 100×200 mm cylinders made from the same sample of concrete and tested at 28 days or at test age designated for f'_c ;
- b. The testing agency performing acceptance testing shall comply with ASTM C1077;
- c. Qualified field testing technicians shall perform tests on fresh concrete at the job site, prepare specimens for standard curing, prepare specimens for field curing, if required, and record the temperature of the fresh concrete when preparing specimens for strength tests;
- d. Qualified laboratory technicians shall perform required laboratory tests;
- e. All reports of acceptance tests shall be provided to the licensed design professional, contractor, concrete producer, and, if requested, to the owner and the building official.

426.12.2 Frequency of Testing

426.12.2.1 Compliance Requirements:

- a. Samples for preparing strength test specimens of each concrete mixture placed each day shall be taken in accordance with (1) through (3):
 - 1. At least once a day;
 - 2. At least once for each 110 m³ of concrete;
 - 3. At least once for each 460 m² of surface area for slabs or walls.
- b. On a given project, if total volume of concrete is such that frequency of testing would provide fewer than five strength tests for a given concrete mixture, strength test specimens shall be made from at least five randomly selected batches or from each batch if fewer than five batches are used;

- c. If the total quantity of a given concrete mixture is less than 38 m^3 , strength tests are not required if evidence of satisfactory strength is submitted to and approved by the building official.

426.12.3 Acceptance Criteria for Standard-Cured Specimens

426.12.3.1 Compliance Requirements:

- a. Specimens for acceptance tests shall be in accordance with (1) and (2):
1. Sampling of concrete for strength test specimens shall be in accordance with ASTM C172M;
 2. Cylinders for strength tests shall be made and standard-cured in accordance with ASTM C31M and tested in accordance with ASTM C39M.
- b. Strength level of a concrete mixture shall be acceptable if (1) and (2) are satisfied:
1. Every arithmetic average of any three consecutive strength tests equals or exceeds f'_c ;
 2. No strength test falls below f'_c by more than 3.5 MPa if f'_c is 35 MPa or less; or by more than $0.10f'_c$ if f'_c exceeds 35 MPa.
- c. If either of the requirements of Section 426.12.3.1(b) are not satisfied, steps shall be taken to increase the average of subsequent strength results;
- d. Requirements for investigating low strength-test results shall apply if the requirements of Section 426.12.3.1(b)(2) are not met.

426.12.4 Investigation of Low Strength-Test Results

426.12.4.1 Compliance Requirements:

- a. If any strength test of standard-cured cylinders falls below f'_c by more than the limit allowed for acceptance, or if tests of field-cured cylinders indicate deficiencies in protection and curing, steps shall be taken to ensure that structural adequacy of the structure is not jeopardized;
- b. If the likelihood of low-strength concrete is confirmed and calculations indicate that structural adequacy is significantly reduced, tests of cores drilled from the area in question in accordance with ASTM C42M shall be permitted. In such cases, three cores shall be taken for each strength test that falls

below f'_c by more than the limit allowed for acceptance;

- c. Cores shall be obtained, moisture-conditioned by storage in watertight bags or containers, transported to the testing agency, and tested in accordance with ASTM C42. Cores shall be tested between 48 hours and 7 days after coring unless otherwise approved by the licensed design professional. The specifier of tests referenced in ASTM C42M shall be the licensed design professional;
- d. Concrete in an area represented by core tests shall be considered structurally adequate if (1) and (2) are satisfied:
1. The average of three cores is equal to at least 85 percent of f'_c ;
 2. No single core is less than 75 percent of f'_c .
- e. Additional testing of cores extracted from locations represented by erratic core strength results shall be permitted;
- f. If criteria of for evaluating structural adequacy based on core strength results are not met, and if the structural adequacy remains in doubt, the responsible authority shall be permitted to order a strength evaluation in accordance with Section 427 for the questionable portion of the structure or take other appropriate action.

426.12.5 Acceptance of Steel Fiber-Reinforced Concrete

426.12.5.1 Compliance Requirements:

- a. Steel fiber-reinforced concrete used for shear resistance shall satisfy (1) through (3):
1. The compressive strength acceptance criteria for standard-cured specimens;
 2. The residual strength obtained from flexural testing in accordance with ASTM C1609M at a mid-span deflection of $1/300$ of the span length is at least the greater of (i) and (ii):
 - i. 90 percent of the measured first-peak strength obtained from a flexural test and;
 - ii. 90 percent of the strength corresponding to $0.62\sqrt{f'_c}$.

3. The residual strength obtained from flexural testing in accordance with ASTM C1609M at a mid-span deflection of 1/150 of the span length is at least the greater of (i) and (ii):

- i. 75 percent of the measured first-peak strength obtained from a flexural test and;
- ii. 75 percent of the strength corresponding to $0.62\sqrt{f'_c}$.

426.13 Inspection

426.13.1 General

426.13.1.1 Concrete construction shall be inspected as required by the general building code.

426.13.1.2 In the absence of general building code inspection requirements, concrete construction shall be inspected throughout the various Work stages by or under the supervision of a licensed design professional or by a qualified inspector in accordance with the provisions of this section.

426.13.1.3 The licensed design professional, a person under the supervision of a licensed design professional, or a qualified inspector shall verify compliance with construction documents.

426.13.1.4 For continuous construction inspection of special moment frames, qualified inspectors under the supervision of the licensed design professional responsible for the structural design or under the supervision of a licensed design professional with demonstrated capability to supervise inspection of these elements shall inspect placement of reinforcement and concrete.

426.13.2 Inspection reports

426.13.2.1 Inspection reports shall document inspected items and be developed throughout each construction Work stage by the licensed design professional, person under the supervision of a licensed design professional, or qualified inspector. Records of the inspection shall be preserved by the party performing the inspection for at least two years after completion of the project.

426.13.2.2 Inspection reports shall document (a) through (e)

- a. General progress of the Work;
- b. Any significant construction loadings on completed floors, members, or walls;

- c. The date and time of mixing, quantity, proportions of materials used, approximate placement location in the structure, and results of tests for fresh and hardened concrete properties for all concrete mixtures used in the Work;

- d. Concrete temperatures and protection given to concrete during placement and curing when the ambient temperature falls below 5 °C or rises above 35°C.

426.13.2.3 Test reports shall be reviewed to verify compliance with Section 420.2.2.5 if ASTM A615M deformed reinforcement is used to resist earthquake-induced flexure, axial forces, or both in special moment frames, special structural walls, and components of special structural walls including coupling beams and wall piers.

426.13.3 Items Requiring Inspection

426.13.3.1 Unless otherwise specified in the general building code, items requiring verification and inspection shall be continuously or periodically inspected in accordance with Sections 426.13.3.2 and 426.13.3.3.

426.13.3.2 Items requiring continuous inspection shall include (a) through (d):

- a. Placement of concrete;
- b. Tensioning of prestressing steel and grouting of bonded tendons;
- c. Installation of adhesive anchors in horizontal or upwardly inclined orientations to resist sustained tension loads in accordance with Section 417.8.2.4;
- d. Reinforcement for special moment frames.

426.13.3.3 Items requiring periodic inspection shall include (a) through (g):

- a. Placement of reinforcement, embedments, and post-tensioning tendons;
- b. Curing method and duration of curing for each member;
- c. Construction and removal of forms and reshoring;
- d. Sequence of erection and connection of precast members;
- e. Verification of in-place concrete strength before stressing post-tensioned reinforcement and before

removal of shores and formwork from beams and structural slabs;

- f. Installation of cast-in anchors, expansion anchors, and undercut anchors in accordance with Section 417.8.2;
- g. Installation of adhesive anchors where continuous inspection is not required in accordance with Section 417.8.2.4 or as a condition of the assessment in accordance with ACI 355.4M.

SECTION 427

STRENGTH EVALUATION OF EXISTING STRUCTURES

427.1 Scope

427.1.1 Provisions of this section shall apply to the strength evaluation of existing structures by analytical means or by load testing.

427.2 General

427.2.1 If there is doubt that a part or all of a structure meets the safety requirements of this Code and the structure is to remain in service, a strength evaluation shall be carried out as required by the licensed design professional or building official.

427.2.2 If the effect of a strength deficiency is well understood and it is practical to measure the dimensions and determine the material properties of the members required for analysis, an analytical evaluation of strength based on this information is permitted. Required data shall be determined in accordance with Section 427.3.

427.2.3 If the effect of a strength deficiency is not well understood or it is not practical to measure the dimensions and determine the material properties of the members required for analysis, a load test is required in accordance with Section 427.4.

427.2.4 If uncertainty about the strength of part or all of a structure involves deterioration, and if the observed response during the load test satisfies the acceptance criteria in Section 427.4.5, the structure or part of the structure is permitted to remain in service for a time period specified by the licensed design professional. If deemed necessary by the licensed design professional, periodic re-evaluations shall be conducted.

427.3 Analytical Strength Evaluation

427.3.1 Verification of As-built Condition

427.3.1.1 Dimensions of members shall be established at critical sections.

427.3.1.2 Locations and sizes of reinforcement shall be determined by measurement. It shall be permitted to base reinforcement locations on available drawings if field-verified at representative locations to confirm the information on the drawings.

427.3.1.3 If required, an estimated equivalent f'_c shall be based on analysis of results of cylinder tests from the original construction or tests of cores removed from the part of the structure where strength is in question.

427.3.1.4 The method for obtaining and testing cores shall be in accordance with ASTM C42M.

427.3.1.5 The properties of reinforcement are permitted to be based on tensile tests of representative samples of the material in the structure.

427.3.2 Strength Reduction Factors

427.3.2.1 If dimensions, size and location of reinforcement, and material properties are determined in accordance with Section 427.3.1, it is permitted to increase ϕ from the design values elsewhere in this Code, however ϕ shall not exceed the limits in Table 427.3.2.1.

Table 427.3.2.1
Maximum Permissible Strength Reduction Factors

Strength	Classification	Transverse reinforcement	Maximum Permissible ϕ
Flexure, Axial, or both	Tension Controlled	All cases	1.0
	Compression Controlled	Spirals*	0.9
		Other	0.8
Shear, Torsion, or both			0.8
Bearing			0.8

* Spirals shall satisfy Sections 410.7.6.3, 420.2.2, and 425.7.3.

427.4 Strength Evaluation by Load Test

427.4.1 General

427.4.1.1 Load tests shall be conducted in a manner that provides for safety of life and the structure during the test.

427.4.1.2 Safety measures shall not interfere with the load test or affect the results.

427.4.1.3 The portion of the structure subject to the test load shall be at least 56 days old. If the owner of the structure, the contractor, the licensed design professional, and all other involved parties agree, it shall be permitted to perform the load test at an earlier age.

427.4.1.4 A precast member to be made composite with cast-in-place concrete shall be permitted to be tested in flexure as a precast member alone in accordance with (a) and (b):

- Test loads shall be applied only when calculations indicate the isolated precast member will not fail by compression or buckling;
- The test load, when applied to the precast member alone, shall induce the same total force in the tensile reinforcement as would be produced by loading the composite member with the test load in accordance with Section 427.4.2.

427.4.2 Test Load Arrangement and Load Factors

427.4.2.1 Test load arrangements shall be selected to maximize the deflection, load effects, and stresses in the critical regions of the members being evaluated.

427.4.2.2 The total test load, T_t , including dead load already in place shall be at least the greatest of (a), (b), and (c):

$$a. \quad T_t = 1.15D + 1.5L + 0.4(L_r \text{ or } R) \quad (427.4.2.2a)$$

$$b. \quad T_t = 1.15D + 0.9L + 1.5(L_r \text{ or } R) \quad (427.4.2.2b)$$

$$c. \quad T_t = 1.3D \quad (427.4.2.2c)$$

427.4.2.3 It is permitted to reduce L in Section 427.4.2.2 in accordance with the general building code.

427.4.2.4 The load factor on the live load L in Section 427.4.2.2(b) shall be permitted to be reduced to 0.45 except for parking structures, areas occupied as places of public assembly, or areas where L is greater than 4.8 kPa.

427.4.3 Test Load Application

427.4.3.1 Total test load, T_t , shall be applied in at least four approximately equal increments.

427.4.3.2 Uniform T_t shall be applied in a manner that ensures uniform distribution of the load transmitted to the structure or portion of the structure being tested. Arching of the test load shall be avoided.

427.4.3.3 After the final load increment is applied, T_t shall remain on the structure for at least 24 hours unless signs of distress, as noted in Section 427.4.5, are observed.

427.4.3.4 After all response measurements are recorded, the test load shall be removed as soon as practical.

427.4.4 Response Measurements

427.4.4.1 Response measurements, such as deflection, strain, slip, and crack width, shall be made at locations where maximum response is expected. Additional measurements shall be made if required.

427.4.4.2 The initial value for all applicable response measurements shall be obtained not more than 1 hour before applying the first load increment.

427.4.4.3 A set of response measurements shall be recorded after each load increment is applied and after T_t has been applied on the structure for at least 24 hours.

427.4.4.4 A set of final response measurements shall be made 24 hours after T_t is removed.

427.4.5 Acceptance Criteria

427.4.5.1 The portion of the structure tested shall show no spalling or crushing of concrete, or other evidence of failure.

427.4.5.2 Members tested shall not exhibit cracks indicating imminent shear failure.

427.4.5.3 In regions of members without transverse reinforcement, structural cracks inclined to the longitudinal axis and having a horizontal projection greater than the depth of the member shall be evaluated. For variable-depth members, the depth shall be measured at the mid-length of the crack.

427.4.5.4 In regions of anchorage and lap splices of reinforcement, short inclined cracks or horizontal cracks along the line of reinforcement shall be evaluated.

427.4.5.5 Measured deflections shall satisfy (a) or (b):

$$\text{a.} \quad \Delta_1 \leq \frac{\ell_t^2}{20,000h} \quad (427.4.5.5a)$$

$$\text{b.} \quad \Delta_r \leq \frac{\Delta_1}{4} \quad (427.4.5.5b)$$

427.4.5.6 If Section 427.4.5.5 is not satisfied, it shall be permitted to repeat the load test provided that the second load test begins no earlier than 72 hours after removal of externally applied loads from the first load test.

427.4.5.7 Portions of the structure tested in the second load test shall be considered acceptable if:

$$\Delta_r \leq \frac{\Delta_2}{5} \quad (427.4.5.7b)$$

427.5 Reduced Load Rating

427.5.1 If the structure under investigation does not satisfy conditions or criteria of Section 427.3 or 427.4.5, the structure shall be permitted for use at a lower load rating, based on the results of the load test or analysis, if approved by the building official.

SECTION 428**BUILDING CODE REQUIREMENTS
FOR CONCRETE THIN SHELLS****428.1 Scope and Definitions**

428.1.1 Provisions of this section shall apply to thin shell and folded plate concrete structures, including ribs and edge members.

428.1.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of Section 428, shall apply to thin-shell structures.

428.1.3 Thin Shells

Three-dimensional spatial structures made up of one or more curved slabs or folded plates whose thicknesses are small compared to their other dimensions. Thin shells are characterized by their three-dimensional load-carrying behavior, which is determined by the geometry of their forms, by the manner in which they are supported, and by the nature of the applied load.

428.1.4 Folded Plates

A class of shell structure formed by joining flat, thin slabs along their edges to create a three-dimensional spatial structure.

428.1.5 Ribbed Shells

Spatial structures with material placed primarily along certain preferred rib lines, with the area between the ribs filled with thin slabs or left open.

428.1.6 Auxiliary Members

Ribs or edge beams that serve to strengthen, stiffen, or support the shell; usually, auxiliary members act jointly with the shell.

428.1.7 Elastic Analysis

An analysis of deformations and internal forces based on equilibrium, compatibility of strains, and assumed elastic behavior, and representing to a suitable approximation the three-dimensional action of the shell together with its auxiliary members.

428.1.8 Inelastic Analysis

An analysis of deformations and internal forces based on equilibrium, nonlinear stress-strain relations for concrete and reinforcement, consideration of cracking and time-

dependent effects, and compatibility of strains. The analysis shall represent to a suitable approximation three-dimensional action of the shell together with its auxiliary members.

428.1.9 Experimental Analysis

An analysis procedure based on the measurement of deformations or strains, or both, of the structure or its model; experimental analysis is based on either elastic or inelastic behavior.

428.2 Analysis and Design

428.2.1 Elastic behavior shall be an accepted basis for determining internal forces and displacements of thin shells. This behavior shall be permitted to be established by computations based on an analysis of the uncracked concrete structure in which the material is assumed linearly elastic, homogeneous and isotropic. Poisson's ratio of concrete shall be permitted to be taken equal to zero.

428.2.2 Inelastic analysis shall be permitted to be used where it can be shown that such methods provide a safe basis for design.

428.2.3 Equilibrium checks of internal resistances and external loads shall be made to ensure consistency of results.

428.2.4 Experimental or numerical analysis procedures shall be permitted where it can be shown that such procedures provide a safe basis for design.

428.2.5 Approximate methods of analysis shall be permitted where it can be shown that such methods provide a safe basis for design.

428.2.6 In prestressed shells, the analysis shall also consider behavior under loads induced during prestressing, at cracking load, and at factored load. Where tendons are draped within a shell, design shall take into account force components on the shell resulting from the tendon profile not lying in one plane.

428.2.7 The thickness of a shell and its reinforcement shall be proportioned for the required strength and serviceability, using either the strength design method of Section 422.1.3 or the design method of Section 424.

428.2.8 Shell instability shall be investigated and shown by design to be precluded.

428.2.9 Auxiliary members shall be designed according to the applicable provisions of this chapter. It

shall be permitted to assume that a portion of the shell equal to the flange width, as specified in Section 409.2.4, acts with the auxiliary member. In such portions of the shell, the reinforcement perpendicular to the auxiliary member shall be at least equal to that required for the flange of a T-beam by Section 409.2.4.4.

428.2.10 Strength design of shell slabs for membrane and bending forces shall be based on the distribution of stresses and strains as determined from either elastic or an inelastic analysis.

428.2.11 In a region where membrane cracking is predicted, the nominal compressive strength parallel to the cracks shall be taken as $0.4f'_c$.

428.3 Design Strength

428.3.1 Specified compressive strength of concrete f'_c at 28 days shall not be less than 21 MPa.

428.3.2 Specified yield strength of non-prestressed reinforcement f_y shall not exceed 420 MPa.

428.4 Specified Cover for Thin Shells Unless a greater concrete cover is required by the General Building Code for fire protection, specified concrete cover shall be in accordance with Section 428.4.1 through 428.4.3. For shells subjected to corrosive environments, 428.4.4 shall apply.

428.4.1 Cast-in-place Non-Prestressed Concrete

Non-prestressed cast-in-place concrete shells shall have specified cover for reinforcement shall be at least that given in Table 428.4.1.

Table 428.4.1
Specified Concrete Cover for Cast-in Place Non-Prestressed Concrete Shells

Concrete exposure	Reinforcement	Specified Cover, mm
Exposed to weather or in contact with the ground	20 mm ϕ or larger	50
	16 mm ϕ , MW200 or MD200 wire, and smaller	40
Not exposed to weather or in contact with the ground	20 mm ϕ or larger	20
	16 mm ϕ , MW200 or MD200 wire, and smaller	12

428.4.2 Cast-in-place Prestressed Concrete

Prestressed cast-in-place concrete shells shall have specified cover for reinforcement, ducts, and end fittings at least that given in Table 428.4.2.

Table 428.4.2
Specified Concrete Cover for Cast-in Place Prestressed Concrete Shells

Concrete exposure	Reinforcement	Specified Cover, mm
Exposed to weather or in contact with the ground	Prestressing tendons and prestressing reinforcement; 25 mm ϕ bar, MW200 or MD 200 wire, and smaller	25
	28 mm ϕ bar and larger	d_h
Not exposed to weather or in contact with the ground	Prestressing tendons and prestressing reinforcement	20
	20 mm ϕ bar and larger	d_b
	16 mm ϕ bar, MW200 or MD200 wire, and smaller	10

428.4.3 Precast Non-Prestressed or Prestressed Concrete Manufactured Under Plant Conditions

Precast non-prestressed concrete manufactured under plant conditions shells shall have specified cover for reinforcement, ducts, and end fittings at least that given in Table 428.4.3.

Table 428.4.3
Specified Concrete Cover for Precast Non-Prestressed or Prestressed Concrete Shells Manufactured Under Plant Conditions

Concrete exposure	Reinforcement	Specified Cover, mm
Exposed to weather or in contact with the ground	20 mm ϕ through 36 mm ϕ bars. Tendons and prestressing reinforcement larger than 16 mm ϕ through 40 mm ϕ	40
	16 mm ϕ bar, MW200 or MD200 wire, and smaller; Tendons and strands 16 mm ϕ and smaller	32
Not exposed to weather or in contact with the ground	Prestressing tendons and prestressing reinforcement	20
	20 mm ϕ bar and larger	16
	16 mm ϕ bar, MW200 or MD200 wire, and smaller	10

428.4.4 Specified Concrete Cover Requirements for Corrosive Environments

428.4.4.1 In corrosive environment or other severe exposure conditions, the specified concrete cover shall be increased as deemed necessary and specified by the licensed design professional. The applicable requirements for concrete based on exposure categories in Section 419.3.1.1 shall be satisfied, or other protection shall be provided.

428.4.4.2 For prestressed concrete members classified as Class T or C in Section 424.4.2.1 and exposed to corrosive environments or other severe exposure categories such as those defined in Section 419.3.1.1, the specified concrete cover shall be at least 1.5 times the cover for prestressed reinforcement required by Section 405.1.2 for cast-in-place prestressed concrete members and Section 405.1.3 for precast concrete members.

428.4.4.3 The increase cover requirement of Section 428.4.4.2 need not be satisfied if the precompressed tensile zone is not in tension under sustained loads.

428.4.5 Concrete Surface Exposed to Earth or Weather

428.4.5.1 If concrete surface is exposed to earth or weather, concrete cover provisions shall be in accordance with Section 420.6.1.

428.5 Shell Reinforcement

428.5.1 Shell reinforcement shall be provided to resist tensile stresses from internal membrane forces, to resist tension from bending and twisting moments, to limit shrinkage and temperature cracking and as reinforcement at shell boundaries, load attachments and shell openings.

428.5.2 Tensile reinforcement shall be provided in two or more directions and shall be proportioned such that its resistance in any direction equals or exceeds the component of internal forces in that direction.

Alternatively, reinforcement for the membrane forces in the slab shall be calculated as the reinforcement required to resist axial tensile forces plus the tensile force due to shear- friction required to transfer shear across any cross section of the membrane. The assumed coefficient of friction, μ , shall not exceed that specified in Section 422.9.4.2.

428.5.3 The minimum area of shell reinforcement at any section as measured in two orthogonal directions shall be at least **0.0018** times the gross area of the section for

Grade 420 reinforcement or **0.0020** for Grade 280 reinforcement.

428.5.4 Reinforcement for shear and bending moments about axes in the plane of the shell slab shall be calculated in accordance with Sections 406, 407 and 408.

428.5.5 The area of shell tension reinforcement shall be limited so that the reinforcement will yield before either crushing of concrete in compression or shell buckling can take place.

428.5.6 In regions of high tension, membrane reinforcement shall, if practical, be placed in the general directions of the principal tensile membrane forces. Where this is not practical, it shall be permitted to place membrane reinforcement in two or more component directions.

428.5.7 If the direction of reinforcement varies more than 10 degrees from the direction of principal tensile membrane force, the amount of reinforcement shall be reviewed in relation to cracking at service loads.

428.5.8 If the magnitude of the principal tensile membrane stress within the shell varies greatly over the area of the shell surface, reinforcement resisting the total tension shall be permitted to be concentrated in the regions of largest tensile stress where it can be shown that this provides a safe basis for design. The ratio of shell reinforcement in any portion of the tensile zone shall be at least **0.0035** based on the overall thickness of the shell.

428.5.9 Reinforcement required to resist shell bending moments shall be proportioned with due regard to the simultaneous action of membrane axial forces at the same location. Where shell reinforcement is required in only one face to resist bending moments, equal amounts shall be placed near both surfaces of the shell even though a reversal of bending moments is not indicated by the analysis.

428.5.10 Shell reinforcement in any direction shall not be spaced farther apart than 450 mm, nor farther apart than five times the shell thickness. Where the principal membrane tensile stress on the gross concrete area due to factored loads exceeds $0.33\phi\lambda\sqrt{f'_c}$ reinforcement shall not be spaced farther apart than three times the shell thickness and 450 mm.

428.5.11 Shell reinforcement at the junction of the shell and supporting members or edge members shall be anchored in or extended through such members in accordance with the requirements of Section 425, except that the minimum development length shall be **1.2 ℓ_d** but not less than 450 mm.

428.5.12 Splice development lengths of shell reinforcement shall be governed by the provisions of Section 425.4.2, except that the minimum splice length of tension bars shall be 1.2 times the value required by Section 425, but not less than 450 mm. The number of splices in principal tensile reinforcement shall be kept to a practical minimum. Where splices are necessary, they shall be staggered at least ℓ_d with not more than one-third of the reinforcement spliced at any section.

428.6 Construction

428.6.1 When removal of formwork is based on a specific modulus of elasticity of concrete because of stability or deflection considerations, the value of the modulus of elasticity E_c shall be determined from flexural tests of field-cured beam specimens. The number of test specimens, the dimensions of test beam specimens, and test procedures shall be specified by the engineer-of-record.

428.6.2 Contract documents shall specify the tolerances for the shape of the shell. If construction results in deviations from the shape greater than the specified tolerances, an analysis of the effect of the deviations shall be made and any required remedial actions shall be taken to ensure safe behavior.

SECTION 429 ALTERNATE DESIGN METHOD

429.1 Notations

See Section 402.2.

429.2 Scope

429.2.1 Non-prestressed reinforced concrete members shall be permitted to be designed using service loads (without load factors) and permissible service load stresses in accordance with provisions of Section 429.

429.2.2 For design of members not covered by Section 424, appropriate provisions of this Code shall apply.

429.2.3 All applicable provisions of this Code for non-prestressed concrete, except Section 406.5, shall apply to members designed by the Alternate Design Method.

429.2.4 Flexural members shall meet requirements for deflection control in Section 424.2.1, and requirements of Sections 408.7.2, 409.2.3, 409.6.1, and 409.9 of this Code.

429.3 General

429.3.1 Load factors and strength reduction factors ϕ shall be taken as unity for members designed by the Alternate Design Method.

429.3.2 It shall be permitted to proportion members for 75 percent of capacities required by other parts of Section 429 when considering wind or earthquake forces combined with other loads, provided the resulting section is not less than that required for the combination of dead and live load.

429.3.3 When dead load reduces effects of other loads, members shall be designed for 85 percent of dead load in combination with the other loads.

429.4 Permissible Service Load Stresses

429.4.1 Stresses in concrete shall not exceed the following:

1. Flexure

Extreme fiber stress in compression $0.45f'_c$

2. Shear*

Beams and one-way slabs and footings:

$$\begin{aligned} \text{Shear carried by concrete, } v_c & \dots\dots\dots \frac{1}{11} \sqrt{f'_c} \\ \text{Maximum shear carried by concrete plus shear} \\ \text{reinforcement, } v_c^{**} & \dots\dots\dots \frac{3}{8} \sqrt{f'_c} \end{aligned}$$

Joists:

$$\text{Shear carried by concrete, } v_c \dots\dots\dots \frac{1}{11} \sqrt{f'_c}$$

Two-way slabs and footings:

$$\begin{aligned} \text{Shear carried by concrete, } \\ v_c & \dots\dots\dots \left(\frac{1}{12} \right) (1 + 2/\beta_c) \sqrt{f'_c} \\ \text{but not greater than } & \dots\dots\dots \frac{1}{6} \sqrt{f'_c} \end{aligned}$$

$$3. \text{ Bearing on loaded area}^\dagger \dots\dots\dots 0.3 f'_c$$

429.4.2 Tensile stress in reinforcement f_s shall not exceed the following:

1. Grade 280 reinforcement 140 MPa
2. Grade 420 reinforcement or greater and welded wire fabric (plain or deformed) 170 MPa
3. For flexural reinforcement, $\phi 10$ mm or less, in one-way slabs of not more than 4 m span but not greater than 200 MPa $0.50 f_y$

429.5 Development and Splices of Reinforcement

429.5.1 Development and splices of reinforcement shall be as required in Section 425 of this chapter.

429.5.2 In satisfying requirements of Section 409.7.3.8.3, M_n shall be taken as computed moment capacity assuming all positive moment tension reinforcement at the section to be stressed to the permissible tensile stress f_s , and V_u shall be taken as unfactored shear force at the section.

* For more detailed calculation of the shear stress carried by concrete v_c and shear values for lightweight aggregate concrete, see Section 429.8.4.

** Designed in accordance with Section 406.4 of this code.

† When the supporting surface is wider on all sides than the loaded area, permissible bearing stress on the loaded area shall be permitted to be multiplied by $\sqrt{A_2/A_1}$ but not more than 2. When the supporting surface is sloped or stepped, A_2 shall be permitted to be taken as the area of the lower base of the largest frustum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical-to 2 horizontal.

429.6 Flexure

For investigation of stresses at service loads, straight-line theory for flexure shall be used with the following assumptions:

429.6.1 Strains vary linearly as the distance from the neutral axis, except for deep flexural members with overall depth-span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a nonlinear distribution of strain shall be considered. See Section 409.9 of this Chapter.

429.6.2 Stress-strain relationship of concrete is a straight line under service loads within permissible service load stresses.

429.6.3 In reinforced concrete members, concrete resists no tension.

429.6.4 It shall be permitted to take the modular ratio, $n = E_s/E_c$, as the nearest whole number (but not less than 6). Except in calculations for deflections, value of n for lightweight concrete shall be assumed to be the same as for normal weight concrete of the same strength.

429.6.5 In doubly reinforced flexural members, an effective modular ratio of $2E_s/E_c$ shall be used to transform compression reinforcement for stress computations. Compressive stress in such reinforcement shall not exceed permissible tensile stress.

429.7 Compression Members With or Without Flexure

429.7.1 Combined flexure and axial load capacity of compression members shall be taken as 40 percent of that computed in accordance with provisions in Section 422 of this Chapter.

429.7.2 Slenderness effects shall be included according to requirements of Table 406.6.3.1.1(b), Sections 406.6.4.4.1 through 406.6.4.5.1. In Eqs. 406.6.4.4.1, 406.6.4.5.1 and 406.6.4.5.2 the term P_u shall be replaced by 2.5 times the design axial load, and the factor 0.75 shall be taken equal to 1.0.

427.7.3 Walls shall be designed in accordance with Section 411 of this Chapter with flexure and axial load capacities taken as 40 percent of that computed using Section 411. In Eq. 411.5.3.1, ϕ shall be taken equal to 1.0.

429.8 Shear and Torsion

429.8.1 Design shear stress v shall be computed by

$$v = V/(b_w d) \quad (429.8.1)$$

where V is design shear force at section considered.

429.8.2 When the reaction, in direction of applied shear, introduces compression into the end regions of a member, sections located less than a distance d from face of support shall be permitted to be designed for the same shear v as that computed at a distance d .

429.8.3 Whenever applicable, effects of torsion, in accordance with provisions of Section 409.5.4, shall be added. Shear and torsional moment strengths provided by concrete and limiting maximum strengths for torsion shall be taken as 55 percent of the values given in Section 409.

429.8.4 Shear Stress Carried by Concrete

429.8.4.1 For members subject to shear and flexure only, shear stress carried by concrete v_c shall not exceed $0.09\sqrt{f'_c}$ unless a more detailed calculation is made in accordance with Section 429.8.4.4.

429.8.4.2 For members subject to axial compression, shear stress carried by concrete v_c , shall not exceed $0.09\sqrt{f'_c}$ unless a more detailed calculation is made in accordance with Section 429.8.4.5.

429.8.4.3 For members subject to significant axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using

$$v_c = 0.09(1 + 0.6 N/A_g)\sqrt{f'_c} \quad (429.8.4.3)$$

Where N is negative for tension. Quantity N/A_g shall be expressed in MPa.

429.8.4.4 For members subject to shear and flexure only, it shall be permitted to compute v_c by

$$v_c = 0.083\sqrt{f'_c} + 9\rho_w Vd/M \quad (429.8.4.4)$$

but shall not exceed $0.14\sqrt{f'_c}$. Quantity Vd/M shall not be taken greater than 1.0, where M is design moment occurring simultaneously with V at section considered.

429.8.4.5 For members subject to axial compression, it shall be permitted to compute v_c by

$$v_c = 0.09(1 + 0.09 N/A_g)\sqrt{f'_c} \quad (429.8.4.5)$$

Quantity N/A_g shall be expressed in MPa.

429.8.4.6 Shear stresses carried by concrete v_c , apply to normal-weight concrete. When lightweight aggregate concrete is used, one of the following modifications shall apply:

When f_{ct} is specified and concrete is proportioned in accordance with Section 426.4.4, $f_{ct}/6.7$ shall be substituted for $\sqrt{f'_c}$ but the value of $f_{ct}/6.7$ shall not exceed $\sqrt{f'_c}$.

1. When f_{ct} is not specified, the value of $\sqrt{f'_c}$ shall be multiplied by 0.75 for "all-lightweight" concrete and by 0.85 for "sand-lightweight" concrete. Linear interpolation shall be permitted when partial sand replacement is used.

429.8.4.7 In determining shear stress carried by concrete v_c , whenever applicable, effects of axial tension due to creep and shrinkage in restrained members shall be included and it shall be permitted to include effects of inclined flexural compression in variable-depth members.

429.8.5 Shear Stress Carried by Shear Reinforcement

429.8.5.1 Types of Shear Reinforcement

Shear reinforcement shall consist of one of the following:

1. Stirrups perpendicular to axis of member;
2. Welded wire fabric with wires located perpendicular to axis of member making an angle of 45 degrees or more with longitudinal tension reinforcement;
3. Longitudinal reinforcement with bent portion making an angle of 30 degrees or more with longitudinal tension reinforcement;
4. Combinations of stirrups and bent longitudinal reinforcement;
5. Spirals.

429.8.5.2 Design yield strength of shear reinforcement shall not exceed 420 MPa.

429.8.5.3 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fiber and shall be anchored at both ends according to Section 425.7.1 of this section to develop design yield strength of reinforcement.

429.8.5.4 Spacing Limits for Shear Reinforcement

429.8.5.4.1 Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/2$, nor 600 mm.

429.8.5.4.2 Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45-degree line, extending toward the reaction from mid-depth of member ($d/2$) to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

429.8.5.4.3 When $(v - v_c)$ exceeds $\frac{1}{6}\sqrt{f'_c}$, maximum spacing given in Sections 429.8.5.4.1 and 429.8.5.4.2 shall be reduced by one-half.

429.8.5.5 Minimum Shear Reinforcement

429.8.5.5.1 A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members where design shear stress v is greater than one-half the permissible shear stress v_c carried by concrete, except:

1. Slabs and footings;
2. Concrete joist construction defined by Section 408.8 of this section;
3. Beam with total depth not greater than 250 mm, 2.5 times thickness of flange, or one-half the width of web, whichever is greatest.

429.8.5.5.2 Minimum shear reinforcement requirements of Section 429.8.5.5.1 shall be permitted to be waived if shown by test that required ultimate flexural and shear strength can be developed when shear reinforcement is omitted.

429.8.5.5.3 Where shear reinforcement is required by Section 429.8.5.5.1 or by analysis, minimum area of shear reinforcement shall be computed by:

$$A_v = b_w s / 3f_y \quad (429.8.5.5.3)$$

where b_w and s are in mm.

429.8.5.6 Design of Shear Reinforcement

429.8.5.6.1 Where design shear stress v exceeds shear stress carried by concrete v_c , shear reinforcement shall be provided in accordance with Sections 429.8.5.6.2 through 429.8.5.6.8.

429.8.5.6.2 When shear reinforcement perpendicular to axis of member is used:

$$A_v = (v - v_c) b_w s / f_y \quad (429.8.5.6.2)$$

429.8.5.6.3 When inclined stirrups are used as shear reinforcement:

$$A_v = \frac{(v - v_c) b_w s}{f_s (\sin \alpha + \cos \alpha)} \quad (429.8.5.6.3)$$

429.8.5.6.4 When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support:

$$A_v = \frac{(v - v_c) b_w d}{f_s \sin \alpha} \quad (429.8.5.6.4)$$

where $(v - v_c)$ shall not exceed $\left(\frac{1}{8}\right)\sqrt{f'_c}$.

429.8.5.6.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, required area shall be computed by Eq. 429.8.5.6.3.

429.8.5.6.6 Only the center three-quarters of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

429.8.5.6.7 When more than one type of shear reinforcement is used to reinforce the same portion of a member, required area shall be computed as the sum of the various types separately. In such computations, v_c shall be included only once.

429.8.5.6.8 Value of $(v - v_c)$ shall not exceed $\left(\frac{3}{8}\right)\sqrt{f'_c}$.

429.8.6 Shear-Friction

Where it is appropriate to consider shear transfer across a given plane, such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times, shear-friction provisions of Section 422.9 of this Chapter shall be permitted to be applied, with limiting maximum stress for shear taken as 55 percent of that given in Section 422.9.4.4. Permissible stress in shear-friction reinforcement shall be that given in Section 429.4.2.

429.8.7 Special Provisions for Slabs and Footings

429.8.7.1 Shear capacity of slabs and footings in the vicinity of concentrated loads or reactions is governed by the more severe of two conditions:

429.8.7.1.1 Beam action for slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from face of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Sections 429.8.1 through 429.8.5.

429.8.7.1.2 Two-way action for slab or footing, with a critical section perpendicular to plane of slab and located so that its perimeter is a minimum, but need not approach closer than $d/2$ to perimeter of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Sections 429.8.7.2 and 429.8.7.3.

429.8.7.2 Design shear stress v shall be computed by

$$v = V/(b_o d) \quad (429.8.7.2)$$

where V and b_o , shall be taken at the critical section defined in Section 429.8.7.1.2.

429.8.7.3 Design shear stress v shall not exceed v_c given by Eq. 429.8.7.3 unless shear reinforcement is provided

$$v_c = \frac{1}{12} \left(1 + \frac{2}{\beta_c} \right) \sqrt{f'_c} \quad (429.8.7.3)$$

but v_c shall not exceed $\left(\frac{1}{6} \right) \sqrt{f'_c} \beta_c$ is the ratio of long side to short side of concentrated load or reaction area. When lightweight aggregate concrete is used, the modifications of Section 429.8.4.6 shall apply.

429.8.7.4 If shear reinforcement consisting of bars or wires is provided in accordance with Section 408.7.6 of this section, v_c shall not exceed $\left(\frac{1}{12} \right) \sqrt{f'_c}$, and v shall not exceed $0.25 \sqrt{f'_c}$.

429.8.7.5 If shear reinforcement consisting of steel I- or C-shaped sections (shearheads) is provided in accordance with Section 408.7.7 of this section, v on the critical section defined in Section 429.8.7.1.2 shall not exceed $0.3 \sqrt{f'_c}$, and v on the critical section defined in Section 422.6.9.8 shall not exceed $\left(\frac{1}{6} \right) \sqrt{f'_c}$. In Eqs. 422.6.9.6 and 422.6.9.7, design shear force V shall be multiplied by 2 and substituted for V_u .

429.8.8 Special Provisions for Other Members

For design of deep flexural members, brackets and corbels, and walls, the special provisions of Section 416.4

of this section shall be used, with shear strengths provided by concrete and limiting maximum strengths for shear taken as 55 percent of the values given in Section 416. In Section 411.5.4.6, the design axial load shall be multiplied by 1.2 if compression and 2.0 if tension, and substituted for N_u .

429.8.9 Composite Concrete Flexural Members

For design of composite concrete flexural members, permissible horizontal shear stress v_h shall not exceed 55 percent of the horizontal shear strengths given in Section 416.4.3.1 of this Chapter.

APPENDIX A

As an aid to users of the NSCP Vol. 1, 7th Edition and ACI Building Code, information on size, areas, and weights of various steel reinforcement is presented

STEEL REINFORCEMENT INFORMATION

ASTM STANDARD							PHILIPPINE STANDARD (SI)		
Bar Size Designation	Nominal Diameter		Nominal Area		Nominal mass		Bar Size Designation	Nominal Area, mm ²	Nominal mass, kg/m
	inch	mm	in ²	mm ²	lb/ft.	kg/m			
3	0.375	9.5	0.11	71	0.376	0.560	10	79	0.618
4	0.500	12.7	0.20	129	0.668	0.994	12	113	0.890
5	0.625	15.9	0.31	199	1.043	1.552	16	201	1.580
6	0.750	19.1	0.44	294	1.502	2.235	20	314	2.465
7	0.875	22.2	0.60	387	2.044	3.042	n.a	n.a	n.a
8	1.000	25.4	0.79	510	2.670	3.973	25	491	3.851
9	1.128	28.7	1.00	645	3.400	5.060	28	616	4.831
10	1.270	32.3	1.27	819	4.303	6.404	32	804	6.310
11	1.410	35.8	1.57	1006	5.313	7.907	36	1019	7.986
14	1.693	43.0	2.25	1452	7.650	11.382	40	1257	10.861
18	2.257	57.3	4.00	2581	13.60	20.240	58	2642	20.729

ASTM Standard Reinforcement Bars

Type*	Nominal diameter, in.	Nominal area, in. ²	Nominal weight, lb/ft
Seven-wire strand (Grade 250)	1/4 (0.250)	0.036	0.122
	5/6 (0.313)	0.058	0.197
	3/8 (0.375)	0.080	0.272
	7/16 (0.438)	0.108	0.367
	1/2 (0.500)	0.144	0.490
	(0.600)	0.216	0.737
	3/8 (0.375)	0.085	0.290
Seven-wire strand (Grade 270)	7/16 (0.438)	0.115	0.390
	1/2 (0.500)	0.153	0.520
	(0.520)	0.167	0.570
	(0.563)	0.192	0.650
	(0.600)	0.217	0.740
	(0.620)	0.231	0.780
	(0.700)	0.294	1.000

Type*	Nominal diameter, in.	Nominal area, in. ²	Nominal weight, lb/ft
Prestressing wire	0.192	0.029	0.098
	0.196	0.030	0.102
	0.250	0.049	0.170
	0.276	0.060	0.204
Prestressing bars (Type I, plain)	3/4	0.44	1.50
	7/8	0.60	2.04
	1	0.78	2.67
	1-1/8	0.99	3.38
	1-1/4	1.23	4.17
	1-3/8	1.48	5.05
	5/8	0.28	0.98
Prestressing bars (Type II, deformed)	3/4	0.42	1.49
	1	0.85	3.01
	1-1/4	1.25	4.39
	1-3/8	1.58	5.56
	1-3/4	2.58	9.10
	2-1/2	5.16	18.20
	3	6.85	24.09

* Available of some strand, wire, and bar size should be investigated in advance.

WRI STANDARD WIRE REINFORCEMENT (Customary Units)*

W & D size		Nominal Diameter, in.	Nominal area, in.	Nominal weight, lb/ft	Area, in ² /ft of width of various spacings								
					Center-to-center spacing, in.								
Plain	Deformed				2	3	4	6	8	10	12	16	18
W45	D45	0.757	0.450	1.530	2.700	1.800	1.350	0.909	0.680	0.540	0.450	0.340	0.300
W31	D31	0.628	0.310	1.054	1.860	1.240	0.930	0.620	0.470	0.370	0.310	0.232	0.207
W30	D30	0.618	0.300	1.020	1.800	1.200	0.900	0.600	0.450	0.360	0.300	0.225	0.200
W28	D28	0.597	0.280	0.952	1.680	1.120	0.840	0.560	0.420	0.330	0.280	0.210	0.187
W26	D26	0.575	0.260	0.884	1.560	1.040	0.780	0.520	0.390	0.310	0.260	0.195	0.173
W24	D24	0.553	0.240	0.816	1.440	0.960	0.720	0.480	0.360	0.280	0.240	0.180	0.160
W22	D22	0.529	0.220	0.748	1.320	0.880	0.660	0.440	0.330	0.260	0.220	0.165	0.147
W20	D20	0.505	0.200	0.680	1.200	0.800	0.600	0.400	0.300	0.240	0.200	0.150	0.133
W18	D18	0.479	0.180	0.612	1.080	0.720	0.540	0.360	0.270	0.216	0.180	0.135	0.120
W16	D16	0.451	0.160	0.544	0.960	0.640	0.480	0.320	0.240	0.192	0.160	0.120	0.107
W14	D14	0.422	0.140	0.476	0.840	0.560	0.420	0.280	0.210	0.168	0.140	0.105	0.093
W12	D12	0.391	0.120	0.408	0.720	0.480	0.360	0.240	0.180	0.144	0.120	0.090	0.080
W11	D11	0.374	0.110	0.374	0.660	0.440	0.330	0.220	0.165	0.132	0.110	0.083	0.073
W10.5	-	0.366	0.105	0.357	0.630	0.420	0.315	0.210	0.157	0.126	0.105	0.071	0.070
W10	D10	0.357	0.100	0.340	0.600	0.400	0.300	0.200	0.150	0.120	0.100	0.080	0.067
W9.5	-	0.348	0.095	0.323	0.570	0.380	0.285	0.190	0.142	0.114	0.095	0.075	0.063
W9	D9	0.338	0.090	0.306	0.540	0.360	0.270	0.180	0.135	0.108	0.090	0.068	0.060
W8.5	-	0.329	0.085	0.289	0.510	0.340	0.255	0.170	0.127	0.102	0.085	0.064	0.057
W8	D8	0.319	0.080	0.272	0.480	0.320	0.240	0.160	0.120	0.096	0.080	0.060	0.053
W7.5	-	0.309	0.075	0.255	0.450	0.300	0.225	0.150	0.112	0.090	0.075	0.056	0.050
W7	D7	0.299	0.070	0.238	0.420	0.280	0.210	0.140	0.105	0.084	0.070	0.053	0.047
W6.5	-	0.288	0.065	0.221	0.390	0.260	0.195	0.130	0.097	0.078	0.065	0.049	0.043
W6	D6	0.276	0.060	0.204	0.360	0.240	0.180	0.120	0.090	0.072	0.060	0.045	0.040
W5.5	-	0.265	0.055	0.187	0.330	0.220	0.165	0.110	0.082	0.066	0.055	0.041	0.037
W5	D5	0.252	0.050	0.170	0.300	0.200	0.150	0.100	0.075	0.060	0.050	0.038	0.033
W4.5	-	0.239	0.045	0.153	0.270	0.180	0.135	0.090	0.067	0.054	0.045	0.034	0.030
W4	D4	0.226	0.040	0.136	0.240	0.160	0.120	0.080	0.060	0.048	0.040	0.030	0.027
W3.5	-	0.211	0.035	0.119	0.210	0.140	0.105	0.070	0.052	0.042	0.035	0.026	0.023
W3	-	0.195	0.030	0.102	0.180	0.120	0.090	0.060	0.045	0.036	0.030	0.023	0.020
W2.9	-	0.192	0.029	0.098	0.174	0.116	0.087	0.058	0.043	0.035	0.029	0.022	0.019
W2.5	-	0.178	0.025	0.085	0.150	0.100	0.075	0.050	0.037	0.030	0.025	0.019	0.017
W2.1	-	0.161	0.020	0.070	0.130	0.084	0.063	0.042	0.032	0.025	0.021	0.016	0.014
W2	-	0.160	0.020	0.068	0.120	0.080	0.060	0.040	0.030	0.024	0.020	0.015	0.013
W1.4	-	0.134	0.014	0.049	0.084	0.056	0.042	0.028	0.028	0.017	0.014	0.011	0.009

* Reference "Structural Welded Wire Reinforcement Manual of Standard Practice," Wire Reinforcement Institute, Hartford, CT, Sixth Edition, Apr., 2001

WRI STANDARD WIRE REINFORCEMENT (Metric Units)*

Equivalent W & D size Customary Units		MW & MD size Metric Units		Nom. Dia., mm.	Nom. area, mm ²	Nominal weight, kg/m	Area, mm ² /m. of width of various spacings									
Plain	Deformed	Plain	Deformed				Center-to-center spacing, mm.									
							51	76	102	152	203	254	305	406	457	
W45	D45	MW290	MD290	19.23	290	2.280	5686	3816	2843	1908	1429	1142	951	714	635	
W31	D31	MW200	MD200	15.96	200	1.570	3922	2632	1961	1316	985	787	656	493	438	
W30	D31	MW194	MD194	15.70	194	1.020	3804	2553	1902	1276	956	764	636	478	425	
W28	D28	MW181	MD181	15.17	181	0.952	3549	2382	1775	1191	892	713	593	446	396	
W26	D26	MW168	MD168	14.61	168	0.884	3294	2211	1647	1105	828	661	551	414	368	
W24	D24	MW155	MD155	14.04	155	0.816	3039	2040	1520	1020	764	610	508	382	339	
W22	D22	MW142	MD142	13.44	142	0.748	2784	1868	1392	934	700	559	466	350	311	
W20	D20	MW129	MD129	12.83	129	1.010	2540	1693	1270	847	635	508	423	318	282	
W18	D18	MW116	MD116	12.17	116	0.911	2286	1524	1143	762	572	457	381	286	254	
W16	D16	MW103	MD103	11.46	103	0.809	2032	1355	1016	677	508	406	339	254	225	
W14	D14	MW90	MD90	10.72	90	0.708	1778	1185	889	593	445	356	296	222	197	
W12	D12	MW77	MD77	9.93	77	0.607	1524	1016	762	508	381	305	254	190	168	
W11	D11	MW71	MD71	9.50	71	0.556	1397	931	699	466	349	279	233	175	155	
W10.5	-	MW68	-	9.30	68	0.531	1334	889	667	445	332	267	222	167	149	
W10	D10	MW65	MD65	9.07	65	0.506	1270	847	635	423	318	254	212	160	142	
W9.5	-	MW61	-	8.84	61	0.481	1207	804	603	402	301	241	201	150	133	
W9	D9	MW58	MD58	8.59	58	0.456	1143	762	572	381	286	229	191	143	127	
W8.5	-	MW55	-	8.36	55	0.430	1080	720	540	360	269	216	180	135	120	
W8	D8	MW52	MD52	8.10	52	0.405	1016	677	508	339	254	203	169	128	114	
W7.5	-	MW48	-	7.85	48	0.379	963	635	476	318	237	191	159	118	105	
W7	D7	MW45	MD45	7.60	45	0.354	889	593	445	296	222	178	148	111	98	
W6.5	-	MW42	-	7.32	42	0.329	826	550	413	275	205	165	138	103	92	
W6	D6	MW39	MD39	7.01	39	0.304	762	508	381	254	191	152	127	96	85	
W5.5	-	MW36	MD36	6.73	36	0.278	699	466	349	233	174	140	116	89	79	
W5	D5	MW32	MD32	6.40	32	0.253	635	423	318	212	159	127	106	78	70	
W4.5	-	MW29	-	6.07	29	0.228	572	381	286	191	142	114	95.3	71	63	
W4	D4	MW26	MD26	5.74	26	0.202	508	339	254	169	127	102	84.7	64	57	
W3.5	-	MW23	-	5.36	23	0.177	445	296	222	148	110	88.9	74.1	56	50	
W3	-	MW19	-	4.95	19	0.152	381	254	191	127	95.3	76.2	63.5	47	42	
W2.5	-	MW16	-	4.52	16	0.126	317	212	159	106	78.3	63.5	52.9	39	35	
W2.1	-	MW14	-	4.11	14	0.106	267	178	133	88.9	65.6	52.9	44.5	34	31	
W2	-	MW13	-	4.06	13	0.101	254	169	127	84.7	63.5	50.8	42.3	32	28	
W	-	MW10	-	3.51	10	0.076	191	127	95.3	63.5	48.4	38.1	31.8	25	22	
W1.4	-	MW9	-	3.40	9	0.071	178	119	88.9	59.3	44.5	36.0	29.6	22	20	

APPENDIX B

Equivalence between SI Metric, MKS-Metric, and U.S. Customary Unit of Non-homogenous Equations in the Code			
Provision Number	SI-metric stress in MPa	mks-metric stress in kgf/cm ²	U.S. Customary units stress in pounds per square inch (psi)
	1 MPa	10 kgf/cm ²	145 psi
	$f'_c = 21$ MPa	$f'_c = 210$ kgf/cm ²	$f'_c = 3,000$ psi
	$f'_c = 28$ MPa	$f'_c = 280$ kgf/cm ²	$f'_c = 4,000$ psi
	$f'_c = 35$ MPa	$f'_c = 350$ kgf/cm ²	$f'_c = 5,000$ psi
	$f'_c = 40$ MPa	$f'_c = 420$ kgf/cm ²	$f'_c = 6,000$ psi
	$f_y = 280$ MPa	$f_y = 2,800$ kgf/cm ²	$f_y = 40,000$ psi
	$f_y = 420$ MPa	$f_y = 4,200$ kgf/cm ²	$f_y = 60,000$ psi
	$f_{pu} = 1725$ MPa	$f_{pu} = 17,600$ kgf/cm ²	$f_{pu} = 250,000$ psi
	$f_{pu} = 1860$ MPa	$f_{pu} = 19,000$ kgf/cm ²	$f_{pu} = 270,000$ psi
	$\sqrt{f'_c}$ in MPa	$3.18\sqrt{f'_c}$ in kgf/cm ²	$12\sqrt{f'_c}$ in psi
	$0.313\sqrt{f'_c}$ in MPa	$\sqrt{f'_c}$ in kgf/cm ²	$3.77\sqrt{f'_c}$ in psi
	$0.083\sqrt{f'_c}$ in MPa	$0.27\sqrt{f'_c}$ in kgf/cm ²	$\sqrt{f'_c}$ in psi
	$0.17\sqrt{f'_c}$ in MPa	$0.53\sqrt{f'_c}$ in kgf/cm ²	$2\sqrt{f'_c}$ in psi
	$M_{2,min} = P_u(1.5 + 0.03h)$	$M_{2,min} = P_u(15 + 0.03h)$	$M_{2,min} = P_u(0.6 + 0.03h)$
406.6.4.5.4	$\left(0.4 + \frac{f_y}{700}\right)$	$\left(0.4 + \frac{f_y}{7000}\right)$	$\left(0.4 + \frac{f_y}{100,000}\right)$
407.3.1.1.1	$(1.65 - 0.0003w_c) \geq 1.09$	$(1.65 - 0.0003w_c) \geq 1.09$	$(1.65 - 0.0003w_c) \geq 1.09$
407.3.1.1.2	$\frac{0.0018 \times 420}{f_y} A_s$	$\frac{0.0018 \times 4200}{f_y} A_s$	$\frac{0.0018 \times 60,000}{f_y} A_s$
407.6.1.1	$0.41 \frac{b_s s}{f_{yt}}$	$4.2 \frac{b_s s}{f_{yt}}$	$60 \frac{b_s s}{f_{yt}}$
407.7.3.5(c)	$h = \frac{\ell_n \left(0.8 + \frac{f_y}{1400}\right)}{36 + 5\beta(\alpha_{fm} - 0.2)} \geq 125$ mm	$h = \frac{\ell_n \left(0.8 + \frac{f_y}{14,000}\right)}{36 + 5\beta(\alpha_{fm} - 0.2)} \geq 12.5$ cm	$h = \frac{\ell_n \left(0.8 + \frac{f_y}{200,000}\right)}{36 + 5\beta(\alpha_{fm} - 0.2)} \geq 5$ in
408.3.1.2(b)(c)	$h = \frac{\ell_n \left(0.8 + \frac{f_y}{1400}\right)}{36 + 9\beta} \geq 90$ mm	$h = \frac{\ell_n \left(0.8 + \frac{f_y}{14,000}\right)}{36 + 9\beta} \geq 9$ cm	$h = \frac{\ell_n \left(0.8 + \frac{f_y}{200,000}\right)}{36 + 9\beta} \geq 3.5$ in
408.3.1.2(d)(e)	$f_t \leq 0.50\sqrt{f'_c}$	$f_t \leq 1.6\sqrt{f'_c}$	$f_t \leq 6\sqrt{f'_c}$
408.3.4.1	$\frac{0.0018 \times 420}{f_y} A_s$	$\frac{0.0018 \times 4200}{f_y} A_s$	$\frac{0.0018 \times 60,000}{f_y} A_s$
408.6.1.1	$0.17\sqrt{f'_c}$	$0.53\sqrt{f'_c}$	$2\sqrt{f'_c}$
408.6.2.3	$0.50\sqrt{f'_c}$	$1.6\sqrt{f'_c}$	$6\sqrt{f'_c}$
408.7.5.6.3.1(a) and (b)	$A_s = \frac{0.37\sqrt{f'_c} b_w d}{f_y}$ $A_s = \frac{2.1 b_w d}{f_y}$	$A_s = \frac{1.2\sqrt{f'_c} b_w d}{f_y}$ $A_s = \frac{21 b_w d}{f_y}$	$A_s = \frac{4.5\sqrt{f'_c} b_w d}{f_y}$ $A_s = \frac{300 b_w d}{f_y}$
408.7.7.1.2	$\phi 0.5\sqrt{f'_c}$	$\phi 1.6\sqrt{f'_c}$	$\phi 6\sqrt{f'_c}$
409.3.1.1.1	$0.4 + \frac{f_y}{700}$	$0.4 + \frac{f_y}{7000}$	$0.4 + \frac{f_y}{100,000}$
409.3.1.1.2	$(1.65 - 0.0003w_c) \geq 1.09$	$(1.65 - 0.0003w_c) \geq 1.09$	$(1.65 - 0.0005w_c) \geq 1.09$
409.6.1.2(a) and (b)	$\frac{0.25\sqrt{f'_c}}{f_y} b_w d$ $\frac{1.4}{f_y} b_w d$	$\frac{0.80\sqrt{f'_c}}{f_y} b_w d$ $\frac{14}{f_y} b_w d$	$\frac{3\sqrt{f'_c}}{f_y} b_w d$ $\frac{200}{f_y} b_w d$
409.6.3.1	$V_u \leq \phi 0.17\sqrt{f'_c} b_w d$	$V_u \leq \phi 0.53\sqrt{f'_c} b_w d$	$V_u \leq \phi 2\sqrt{f'_c} b_w d$
409.6.3.3(a) and (b)	$A_{v,min} \geq 0.062\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 0.35\sqrt{f'_c} \frac{b_w s}{f_{yt}}$	$A_{v,min} \geq 0.2\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 3.5\sqrt{f'_c} \frac{b_w s}{f_{yt}}$	$A_{v,min} \geq 0.75\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq \frac{50 b_w s}{f_{yt}}$

409.6.4.2(a) and (b)	$(A_v + 2A_t)/s \geq 0.062\sqrt{f'_c} \frac{b_w}{f_{yt}}$ $(A_v + 2A_t)/s \geq \frac{0.35b_w}{f_{yt}}$	$(A_v + 2A_t)/s \geq 0.2\sqrt{f'_c} \frac{b_w}{f_{yt}}$ $(A_v + 2A_t)/s \geq \frac{3.5b_w}{f_{yt}}$	$(A_v + 2A_t)/s \geq 0.75\sqrt{f'_c} \frac{b_w}{f_{yt}}$ $(A_v + 2A_t)/s \geq \frac{50b_w}{f_{yt}}$
409.6.4.3(a) and (b)	$A_{e,min} \leq \frac{0.42\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{A_t}{s}\right)p_h \frac{f_{yt}}{f_y}$ $A_{e,min} \leq \frac{0.42\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{0.175b_w}{f_{yt}}\right)p_h \frac{f_{yt}}{f_y}$	$A_{e,min} \leq \frac{1.33\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{A_t}{s}\right)p_h \frac{f_{yt}}{f_y}$ $A_{e,min} \leq \frac{1.33\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{25b_w}{f_{yt}}\right)p_h \frac{f_{yt}}{f_y}$	$A_{e,min} \leq \frac{5\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{A_t}{s}\right)p_h \frac{f_{yt}}{f_y}$ $A_{e,min} \leq \frac{5\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{25b_w}{f_{yt}}\right)p_h \frac{f_{yt}}{f_y}$
409.7.3.5(c)	$0.41 \frac{b_w s}{f_{yt}}$	$4.2 \frac{b_w s}{f_{yt}}$	$60 \frac{b_w s}{f_{yt}}$
409.7.6.2.2	$0.33\sqrt{f'_c}b_w d$	$1.1\sqrt{f'_c}b_w d$	$4\sqrt{f'_c}b_w d$
409.9.2.1	$V_u \leq \phi 0.83\sqrt{f'_c}b_w d$	$V_u \leq \phi 2.65\sqrt{f'_c}b_w d$	$V_u \leq \phi 10\sqrt{f'_c}b_w d$
410.6.2.2	$A_{v,min} \geq 0.062\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 0.35\sqrt{f'_c} \frac{b_w s}{f_{yt}}$	$A_{v,min} \geq 0.2\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 3.5\sqrt{f'_c} \frac{b_w s}{f_{yt}}$	$A_{v,min} \geq 0.75\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 0.35\sqrt{f'_c} \frac{b_w s}{f_{yt}}$
410.7.6.5.2	$0.33\sqrt{f'_c}b_w d$	$1.1\sqrt{f'_c}b_w d$	$4\sqrt{f'_c}b_w d$
411.5.4.3	$0.83\sqrt{f'_c}hd$	$2.65\sqrt{f'_c}hd$	$10\sqrt{f'_c}hd$
411.5.4.5 and 411.5.4.6(a)	$0.17\lambda\sqrt{f'_c}hd$	$0.53\lambda\sqrt{f'_c}hd$	$2\lambda\sqrt{f'_c}hd$
411.5.4.6(b)	$0.17\left(1 + \frac{0.29N_u}{A_g}\right)\lambda\sqrt{f'_c}b_w d$	$0.53\left(1 + \frac{N_u}{35A_g}\right)\lambda\sqrt{f'_c}b_w d$	$2\left(1 + \frac{N_u}{500A_g}\right)\lambda\sqrt{f'_c}b_w d$
411.5.4.6(d)	$V_c = 0.27\lambda\sqrt{f'_c}hd + \frac{N_u d}{4\ell_w}$	$V_c = 0.88\lambda\sqrt{f'_c}hd + \frac{N_u d}{4\ell_w}$	$V_c = 3.3\lambda\sqrt{f'_c}hd + \frac{N_u d}{4\ell_w}$
411.5.4.6(e)	$V_c = \left[0.05\lambda\sqrt{f'_c} + \frac{\ell_w \left(0.1\lambda\sqrt{f'_c} + \frac{0.2N_u}{\ell_w h}\right)}{\frac{M_u}{V_u} - \frac{\ell_w}{2}}\right]hd$	$V_c = \left[0.16\lambda\sqrt{f'_c} + \frac{\ell_w \left(0.33\lambda\sqrt{f'_c} + \frac{0.2N_u}{\ell_w h}\right)}{\frac{M_u}{V_u} - \frac{\ell_w}{2}}\right]hd$	$V_c = \left[0.16\lambda\sqrt{f'_c} + \frac{\ell_w \left(0.33\lambda\sqrt{f'_c} + \frac{0.2N_u}{\ell_w h}\right)}{\frac{M_u}{V_u} - \frac{\ell_w}{2}}\right]hd$
412.5.3.3	$\sqrt{f'_c} \leq 8.3$ in MPa	$\sqrt{f'_c} \leq 27$ in kgf/cm ²	$\sqrt{f'_c} \leq 100$ in psi
412.5.3.4	$V_u \leq \phi 0.66A_{cv}\sqrt{f'_c}$ $\sqrt{f'_c} \leq 8.3$ in MPa	$V_u \leq \phi 2.1A_{cv}\sqrt{f'_c}$ $\sqrt{f'_c} \leq 27$ in kgf/cm ²	$V_u \leq \phi 8A_{cv}\sqrt{f'_c}$ $\sqrt{f'_c} \leq 100$ in psi
414.5.2.1a	$M_n = 0.42\lambda\sqrt{f'_c}S_m$	$M_n = 1.33\lambda\sqrt{f'_c}S_m$	$M_n = 5\lambda\sqrt{f'_c}S_m$
414.5.4.1(a)	$\frac{M_u}{S_m} - \frac{P_u}{A_g} \leq \phi 0.42\lambda\sqrt{f'_c}$	$\frac{M_u}{S_m} - \frac{P_u}{A_g} \leq \phi 1.33\lambda\sqrt{f'_c}$	$\frac{M_u}{S_m} - \frac{P_u}{A_g} \leq \phi 5\lambda\sqrt{f'_c}$
414.5.5.1(a)	$V_n = 0.11\lambda\sqrt{f'_c}b_w h$	$V_n = 0.35\lambda\sqrt{f'_c}b_w h$	$V_n = \frac{4}{3}\lambda\sqrt{f'_c}b_w h$
414.5.5.1(b) and (c)	$V_n = 0.11\left[1 + \frac{2}{\beta}\right]\lambda\sqrt{f'_c}b_o h$ $V_n = 0.22\lambda\sqrt{f'_c}b_o h$	$V_n = 0.35\left[1 + \frac{2}{\beta}\right]\lambda\sqrt{f'_c}b_o h$ $V_n = 0.71\lambda\sqrt{f'_c}b_o h$	$V_n = \left[1 + \frac{2}{\beta}\right]\frac{4}{3}\lambda\sqrt{f'_c}b_o h$ $V_n = 2\left(\frac{4}{3}\lambda\sqrt{f'_c}b_o h\right)$
415.4.2	$A_{v,min} \geq 0.062\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 0.35\sqrt{f'_c} \frac{b_w s}{f_{yt}}$	$A_{v,min} \geq 0.2\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 3.5\sqrt{f'_c} \frac{b_w s}{f_{yt}}$	$A_{v,min} \geq 0.75\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 50 \frac{b_w s}{f_{yt}}$
416.4.4.1	$\phi(3.5b_v d)$	$\phi(35b_v d)$	$\phi(500b_v d)$
416.4.4.2	$\lambda\left(1.8 + 0.6\frac{A_v f_{yt}}{b_v s}\right)b_v d$ $3.5b_v d$ $0.55b_v d$	$\lambda\left(18 + 0.6\frac{A_v f_{yt}}{b_v s}\right)b_v d$ $35b_v d$ $5.6b_v d$	$\lambda\left(260 + 0.6\frac{A_v f_{yt}}{b_v s}\right)b_v d$ $500b_v d$ $80b_v d$
416.4.6.1	$A_{v,min} \geq 0.062\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 0.35 \frac{b_w s}{f_{yt}}$	$A_{v,min} \geq 0.2\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 3.5 \frac{b_w s}{f_{yt}}$	$A_{v,min} \geq 0.75\sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 50 \frac{b_w s}{f_{yt}}$
416.5.2.4(b) and (c)	$(3.3 + 0.08f'_c)b_w d$ $11b_w d$	$(34 + 0.08f'_c)b_w d$ $110b_w d$	$(480 + 0.08f'_c)b_w d$ $1600b_w d$
416.5.2.5(b)	$\left(5.5 - 1.9\frac{a_v}{d}\right)b_w d$	$\left(55 - 20\frac{a_v}{d}\right)b_w d$	$\left(800 - 280\frac{a_v}{d}\right)b_w d$
417.4.2.2a	$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ $k_c = 10$ or 7	$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ $k_c = 10$ or 7	$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ $k_c = 24$ or 17
417.4.2.2b	$N_b = 3.9\lambda_a \sqrt{f'_c} h_{ef}^{5/3}$	$N_b = 5.8\lambda_a \sqrt{f'_c} h_{ef}^{5/3}$	$N_b = 16\lambda_a \sqrt{f'_c} h_{ef}^{5/3}$
417.4.4.1	$N_{sb} = 13c_{a1}\sqrt{A_{brg}\lambda_a\sqrt{f'_c}}$	$N_{sb} = 42.5c_{a1}\sqrt{A_{brg}\lambda_a\sqrt{f'_c}}$	$N_{sb} = 160c_{a1}\sqrt{A_{brg}\lambda_a\sqrt{f'_c}}$

417.4.5.1d	$10 \sqrt{\frac{\tau_{cr}}{7.6}}$	$10 d_a \sqrt{\frac{\tau_{cr}}{76}}$	$10 d_a \sqrt{\frac{\tau_{cr}}{1100}}$
417.5.2.2a	$V_b = 0.6 \left(\frac{\ell_e}{d_a} \right)^{0.2} \sqrt{d_a \lambda_a \sqrt{f'_c} (c_{a1})^{1.5}}$	$V_b = 0.6 \left(\frac{\ell_e}{d_a} \right)^{0.2} \sqrt{d_a \lambda_a \sqrt{f'_c} (c_{a1})^{1.5}}$	$V_b = 7 \left(\frac{\ell_e}{d_a} \right)^{0.2} \sqrt{d_a \lambda_a \sqrt{f'_c} (c_{a1})^{1.5}}$
417.5.2.2b	$V_b = 3.7 \lambda_a \sqrt{f'_c} (c_{a1})^{1.5}$	$V_b = 3.8 \lambda_a \sqrt{f'_c} (c_{a1})^{1.5}$	$V_b = 9 \lambda_a \sqrt{f'_c} (c_{a1})^{1.5}$
417.5.2.3	$V_b = 0.66 \left(\frac{\ell_e}{d_a} \right)^{0.2} \sqrt{d_a \lambda_a \sqrt{f'_c} (c_{a1})^{1.5}}$	$V_b = 2.1 \left(\frac{\ell_e}{d_a} \right)^{0.2} \sqrt{d_a \lambda_a \sqrt{f'_c} (c_{a1})^{1.5}}$	$V_b = 8 \left(\frac{\ell_e}{d_a} \right)^{0.2} \sqrt{d_a \lambda_a \sqrt{f'_c} (c_{a1})^{1.5}}$
418.7.5.2	$f'_c > 70 \text{ MPa}$	$f'_c > 700 \text{ kgf/cm}^2$	$f'_c > 10,000 \text{ psi}$
418.7.5.3	$s_o = 100 + \left(\frac{350 - h_x}{3} \right)$	$s_o = 10 + \left(\frac{35 - h_x}{3} \right)$	$s_o = 4 + \left(\frac{14 - h_x}{3} \right)$
418.7.5.4(a)	$k_f = \frac{f'_c}{175} + 0.6 \geq 1.0$	$k_f = \frac{f'_c}{1750} + 0.6 \geq 1.0$	$k_f = \frac{f'_c}{25,000} + 0.6 \geq 1.0$
418.8.4.1	$1.7 \lambda \sqrt{f'_c} A_j$ $1.2 \lambda \sqrt{f'_c} A_j$ $1.0 \lambda \sqrt{f'_c} A_j$	$5.3 \lambda \sqrt{f'_c} A_j$ $4.0 \lambda \sqrt{f'_c} A_j$ $3.2 \lambda \sqrt{f'_c} A_j$	$20 \lambda \sqrt{f'_c} A_j$ $15 \lambda \sqrt{f'_c} A_j$ $12 \lambda \sqrt{f'_c} A_j$
418.8.5.1	$\ell_{dh} = f_y d_b / (5.4 \lambda \sqrt{f'_c})$	$\ell_{dh} = f_y d_b / (17 \lambda \sqrt{f'_c})$	$\ell_{dh} = f_y d_b / (65 \lambda \sqrt{f'_c})$
418.10.2.1	$0.083 A_{cv} \lambda \sqrt{f'_c}$	$0.27 A_{cv} \lambda \sqrt{f'_c}$	$A_{cv} \lambda \sqrt{f'_c}$
418.10.2.2	$0.17 A_{cv} \lambda \sqrt{f'_c}$	$0.53 A_{cv} \lambda \sqrt{f'_c}$	$2 A_{cv} \lambda \sqrt{f'_c}$
418.10.4.1	$V_n = A_{cv} (\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$ $\alpha_c = 0.25$ for $\frac{h_w}{\ell_w} \leq 1.5$ $\alpha_c = 0.17$ for $\frac{h_w}{\ell_w} \leq 2.0$	$V_n = A_{cv} (\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$ $\alpha_c = 0.80$ for $\frac{h_w}{\ell_w} \leq 1.5$ $\alpha_c = 0.53$ for $\frac{h_w}{\ell_w} \leq 2.0$	$V_n = A_{cv} (\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$ $\alpha_c = 3.0$ for $\frac{h_w}{\ell_w} \leq 1.5$ $\alpha_c = 0.20$ for $\frac{h_w}{\ell_w} \leq 2.0$
418.10.4.4	$0.66 A_{cv} \lambda \sqrt{f'_c}$ $0.83 A_{cv} \lambda \sqrt{f'_c}$	$2.12 A_{cv} \lambda \sqrt{f'_c}$ $2.65 A_{cv} \lambda \sqrt{f'_c}$	$8 A_{cv} \lambda \sqrt{f'_c}$ $10 A_{cv} \lambda \sqrt{f'_c}$
418.10.4.5	$0.83 A_{cv} \lambda \sqrt{f'_c}$	$2.65 A_{cv} \lambda \sqrt{f'_c}$	$10 A_{cv} \lambda \sqrt{f'_c}$
418.10.6.5(a)	$2.8 / f_y$	$28 / f_y$	$400 / f_y$
418.10.6.5(b)	$0.83 A_{cv} \lambda \sqrt{f'_c}$	$0.27 A_{cv} \lambda \sqrt{f'_c}$	$A_{cv} \lambda \sqrt{f'_c}$
418.10.7.2	$0.33 \lambda \sqrt{f'_c} A_{cw}$	$1.1 \lambda \sqrt{f'_c} A_{cw}$	$4 \lambda \sqrt{f'_c} A_{cw}$
418.10.7.4	$V_n = 2 A_{vd} f_y \sin \alpha \leq 0.83 \sqrt{f'_c} A_{cw}$	$V_n = 2 A_{vd} f_y \sin \alpha \leq 2.65 \sqrt{f'_c} A_{cw}$	$V_n = 2 A_{vd} f_y \sin \alpha \leq 10 \sqrt{f'_c} A_{cw}$
418.12.7.6(b)	$A_{v,min} \geq 0.062 \sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 0.35 \frac{b_w s}{f_{yt}}$	$A_{v,min} \geq 0.2 \sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq 3.5 \frac{b_w s}{f_{yt}}$	$A_{v,min} \geq 0.75 \sqrt{f'_c} \frac{b_w s}{f_{yt}}$ $A_{v,min} \geq \frac{50 b_w s}{f_{yt}}$
418.12.9.1	$V_n = A_{cv} (0.17 \lambda \sqrt{f'_c} + \rho_t f_y)$	$V_n = A_{cv} (0.53 \lambda \sqrt{f'_c} + \rho_t f_y)$	$V_n = A_{cv} (2 \lambda \sqrt{f'_c} + \rho_t f_y)$
418.12.9.2	$0.66 A_{cv} \sqrt{f'_c}$	$2.12 A_{cv} \sqrt{f'_c}$	$8 A_{cv} \sqrt{f'_c}$
418.14.5.1	$0.29 \sqrt{f'_c}$	$0.93 \sqrt{f'_c}$	$3.5 \sqrt{f'_c}$
419.2.2.1(a)	$E_c = w_c^{1.5} 0.43 \sqrt{f'_c}$	$E_c = w_c^{1.5} 0.14 \sqrt{f'_c}$	$E_c = w_c^{1.5} 0.33 \sqrt{f'_c}$
419.2.2.1(b)	$E_c = 4700 \sqrt{f'_c}$	$E_c = 15,100 \sqrt{f'_c}$	$E_c = 57,000 \sqrt{f'_c}$
419.2.3.1	$f_r = 0.62 \lambda \sqrt{f'_c}$	$f_r = 2 \lambda \sqrt{f'_c}$	$f_r = 7.5 \lambda \sqrt{f'_c}$
419.2.4.3	$\lambda = f_{cr} / (0.56 \sqrt{f_{cm}}) \leq 1.0$	$\lambda = f_{cr} / (0.56 \sqrt{f_{cm}}) \leq 1.0$	$\lambda = f_{cr} / (6.7 \sqrt{f_{cm}}) \leq 1.0$
420.3.2.4.1	$f_{se} + 70 + \frac{f'_c}{100 \rho_p}$ $f_{se} + 420$ $f_{ps} = f_{se} + 70 + \frac{f'_c}{300 \rho_p}$ $f_{se} + 210$	$f_{se} + 700 + \frac{f'_c}{100 \rho_p}$ $f_{se} + 4200$ $f_{ps} = f_{se} + 700 + \frac{f'_c}{300 \rho_p}$ $f_{se} + 2100$	$f_{se} + 10,000 + \frac{f'_c}{100 \rho_p}$ $f_{se} + 60,000$ $f_{ps} = f_{se} + 10,000 + \frac{f'_c}{300 \rho_p}$ $f_{se} + 30,000$
421.2.3	$\ell_{tr} = \left(\frac{f_{se}}{21} \right) d_b$	$\ell_{tr} = \left(\frac{f_{se}}{210} \right) d_b$	$\ell_{tr} = \left(\frac{f_{se}}{3000} \right) d_b$
422.2.2.4.3	$0.85 - \frac{0.05(f'_c - 28)}{7}$	$0.85 - \frac{0.05(f'_c - 280)}{70}$	$0.85 - \frac{0.05(f'_c - 4000)}{1000}$
422.5.1.2	$V_u \leq \phi (V_c + 0.66 \sqrt{f'_c} b_w d)$ $V_c = 0.17 \lambda \sqrt{f'_c} b_w d$ $V_c = \left(0.16 \lambda \sqrt{f'_c} + 17 \rho_w \frac{V_u d}{M_u} \right) b_w d$ $\leq \left(0.16 \lambda \sqrt{f'_c} + 17 \rho_w \frac{V_u d}{M_u} \right) b_w d$ $\leq 0.29 \lambda \sqrt{f'_c} b_w d$	$V_u \leq \phi (V_c + 2.2 \sqrt{f'_c} b_w d)$ $V_c = 0.53 \lambda \sqrt{f'_c} b_w d$ $V_c = \left(0.5 \lambda \sqrt{f'_c} + 176 \rho_w \frac{V_u d}{M_u} \right) b_w d$ $\leq \left(0.5 \lambda \sqrt{f'_c} + 176 \rho_w \frac{V_u d}{M_u} \right) b_w d$ $\leq 0.93 \lambda \sqrt{f'_c} b_w d$	$V_u \leq \phi (V_c + 8 \sqrt{f'_c} b_w d)$ $V_c = 2 \lambda \sqrt{f'_c} b_w d$ $V_c = \left(0.19 \lambda \sqrt{f'_c} + 2500 \rho_w \frac{V_u d}{M_u} \right) b_w d$ $\leq \left(0.19 \lambda \sqrt{f'_c} + 2500 \rho_w \frac{V_u d}{M_u} \right) b_w d$ $\leq 3.5 \lambda \sqrt{f'_c} b_w d$

422.5.6.1	$V_c = 0.17 \left(1 + \frac{N_u}{14A_g} \right) \lambda \sqrt{f'_c} b_w d$	$V_c = 0.53 \left(1 + \frac{N_u}{140A_g} \right) \lambda \sqrt{f'_c} b_w d$	$V_c = 2 \left(1 + \frac{N_u}{2000A_g} \right) \lambda \sqrt{f'_c} b_w d$
422.5.6.1(a)	$V_c = \left(0.16 \lambda \sqrt{f'_c} + 17 \rho_w \frac{V_d}{M_n - N_u \frac{4h-d}{8}} \right) b_w d$	$V_c = \left(0.5 \lambda \sqrt{f'_c} + 176 \rho_w \frac{V_d}{M_n - N_u \frac{4h-d}{8}} \right) b_w d$	$V_c = \left(1.9 \lambda \sqrt{f'_c} + 2500 \rho_w \frac{V_d}{M_n - N_u \frac{4h-d}{8}} \right) b_w d$
422.5.6.1(b)	$V_c = 0.29 \lambda \sqrt{f'_c} b_w d \sqrt{1 + \frac{0.29 N_u}{A_g}}$	$V_c = 0.93 \lambda \sqrt{f'_c} b_w d \sqrt{1 + \frac{N_u}{35A_g}}$	$V_c = 3.5 \lambda \sqrt{f'_c} b_w d \sqrt{1 + \frac{N_u}{500A_g}}$
422.5.7.1	$V_c = 0.17 \left(1 + \frac{0.29 N_u}{A_g} \right) \lambda \sqrt{f'_c} b_w d \geq 0$	$V_c = 0.53 \left(1 + \frac{N_u}{35A_g} \right) \lambda \sqrt{f'_c} b_w d \geq 0$	$V_c = 2 \left(1 + \frac{N_u}{500A_g} \right) \lambda \sqrt{f'_c} b_w d \geq 0$
422.5.8.2	$V_c = \left(0.05 \lambda \sqrt{f'_c} + 48 \frac{V_u d_p}{M_u} \right) b_w d$ $V_c \leq (0.05 \lambda \sqrt{f'_c} + 4.8) b_w d$ $0.17 \lambda \sqrt{f'_c} b_w d \leq V_c \leq 0.42 \lambda \sqrt{f'_c} b_w d$	$V_c = \left(0.16 \lambda \sqrt{f'_c} + 49 \frac{V_u d_p}{M_u} \right) b_w d$ $V_c \leq (0.16 \lambda \sqrt{f'_c} + 49) b_w d$ $0.53 \lambda \sqrt{f'_c} b_w d \leq V_c \leq 1.33 \lambda \sqrt{f'_c} b_w d$	$V_c = \left(0.6 \lambda \sqrt{f'_c} + 700 \frac{V_u d_p}{M_u} \right) b_w d$ $V_c \leq (0.6 \lambda \sqrt{f'_c} + 700) b_w d$ $2 \lambda \sqrt{f'_c} b_w d \leq V_c \leq 5 \lambda \sqrt{f'_c} b_w d$
422.5.8.3.1a	$V_{ci} = 0.05 \lambda \sqrt{f'_c} b_w d_p + V_d + \frac{V_i M_{cre}}{M_{max}}$	$V_{ci} = 0.16 \lambda \sqrt{f'_c} b_w d_p + V_d + \frac{V_i M_{cre}}{M_{max}}$	$V_{ci} = 0.6 \lambda \sqrt{f'_c} b_w d_p + V_d + \frac{V_i M_{cre}}{M_{max}}$
422.5.8.3.1b	$V_{ci} = 0.14 \lambda \sqrt{f'_c} b_w d$	$V_{ci} = 0.45 \lambda \sqrt{f'_c} b_w d$	$V_{ci} = 1.7 \lambda \sqrt{f'_c} b_w d$
422.5.8.3.1c	$M_{cre} = \left(\frac{I}{y_t} \right) (0.5 \lambda \sqrt{f'_c} + f_{pe} - f_d)$	$M_{cre} = \left(\frac{I}{y_t} \right) (1.6 \lambda \sqrt{f'_c} + f_{pe} - f_d)$	$M_{cre} = \left(\frac{I}{y_t} \right) (6 \lambda \sqrt{f'_c} + f_{pe} - f_d)$
422.5.8.3.2	$V_{cw} = (0.29 \lambda \sqrt{f'_c} + 0.3 f_{pc}) b_w d_p + V_p$	$V_{cw} = (0.93 \lambda \sqrt{f'_c} + 0.3 f_{pc}) b_w d_p + V_p$	$V_{cw} = (3.5 \lambda \sqrt{f'_c} + 0.3 f_{pc}) b_w d_p + V_p$
422.5.8.3.3	$0.33 \lambda \sqrt{f'_c}$	$1.1 \lambda \sqrt{f'_c}$	$4 \lambda \sqrt{f'_c}$
422.5.10.6.2a	$V_s \leq 0.25 \lambda \sqrt{f'_c} b_w d$	$V_s \leq 0.8 \lambda \sqrt{f'_c} b_w d$	$V_s \leq 3 \lambda \sqrt{f'_c} b_w d$
422.5.10.6.2b			
422.6.3.1	$\sqrt{f'_c} \leq 8.3$ in MPa	$3.18 \sqrt{f'_c} \leq 27$ in kgf/cm ²	$\sqrt{f'_c} \leq 100$ in psi
422.6.5.2(a)	$v_c = 0.33 \lambda \sqrt{f'_c}$	$v_c = 1.1 \lambda \sqrt{f'_c}$	$v_c = 4 \lambda \sqrt{f'_c}$
422.6.5.2(b)	$V_c = 0.17 \left(1 + \frac{2}{\beta} \right) \lambda \sqrt{f'_c}$	$V_c = 0.53 \left(1 + \frac{2}{\beta} \right) \lambda \sqrt{f'_c}$	$V_c = \left(2 + \frac{4}{\beta} \right) \lambda \sqrt{f'_c}$
422.6.5.2(c)	$V_c = 0.083 \left(2 + \frac{\alpha_s d}{b_o} \right) \lambda \sqrt{f'_c}$	$V_c = 0.27 \left(2 + \frac{\alpha_s d}{b_o} \right) \lambda \sqrt{f'_c}$	$V_c = \left(2 + \frac{\alpha_s d}{b_o} \right) \lambda \sqrt{f'_c}$
422.6.5.5	$\sqrt{f'_c} \leq 5.8$ in MPa 0.9 MPa $\leq f_{pe} \leq 3.5$ MPa	$3.18 \sqrt{f'_c} \leq 19$ in kgf/cm ² 9 kgf/cm ² $\leq f_{pe} \leq 35$ kgf/cm ²	$\sqrt{f'_c} \leq 70$ in psi 125 psi $\leq f_{pe} \leq 500$ psi
422.6.5.5a	$v_c = (0.29 \lambda \sqrt{f'_c} + 0.3 f_{pc}) + V_p / (b_o d)$	$v_c = (0.93 \lambda \sqrt{f'_c} + 0.3 f_{pc}) + V_p / (b_o d)$	$v_c = (3.5 \lambda \sqrt{f'_c} + 0.3 f_{pc}) + V_p / (b_o d)$
422.6.5.5b	$v_c = 0.083 \left(1.5 + \frac{\alpha_s d}{b_o} \right) \lambda \sqrt{f'_c}$ $+ 0.3 f_{pc} + V_p / (b_o d)$	$v_c = 0.27 \left(1.5 + \frac{\alpha_s d}{b_o} \right) \lambda \sqrt{f'_c}$ $+ 0.3 f_{pc} + V_p / (b_o d)$	$v_c = \left(1.5 + \frac{\alpha_s d}{b_o} \right) \lambda \sqrt{f'_c}$ $+ 0.3 f_{pc} + V_p / (b_o d)$
422.6.6.1(a), (b), (d)	$0.17 \lambda \sqrt{f'_c}$	$0.53 \lambda \sqrt{f'_c}$	$2 \lambda \sqrt{f'_c}$
422.6.6.1(c)	$0.25 \lambda \sqrt{f'_c}$	$0.80 \lambda \sqrt{f'_c}$	$3 \lambda \sqrt{f'_c}$
422.6.6.2(a)	$0.5 \lambda \sqrt{f'_c}$	$1.6 \lambda \sqrt{f'_c}$	$6 \lambda \sqrt{f'_c}$
422.6.6.2(b)	$\phi 0.66 \sqrt{f'_c}$	$\phi 2.1 \sqrt{f'_c}$	$\phi 8 \sqrt{f'_c}$
422.6.8.3	$\left(\frac{A_v}{s} \right) \geq 0.17 \sqrt{f'_c} \left(\frac{b_o}{f_{yt}} \right)$	$\left(\frac{A_v}{s} \right) \geq 0.53 \sqrt{f'_c} \left(\frac{b_o}{f_{yt}} \right)$	$\left(\frac{A_v}{s} \right) \geq 2 \sqrt{f'_c} \left(\frac{b_o}{f_{yt}} \right)$
422.6.9.10	$\phi 0.33 \sqrt{f'_c}$ $\phi 0.58 \sqrt{f'_c}$	$\phi 1.1 \sqrt{f'_c}$ $\phi 1.9 \sqrt{f'_c}$	$\phi 4 \sqrt{f'_c}$ $\phi 7 \sqrt{f'_c}$
422.6.9.12	$\phi 0.33 \sqrt{f'_c}$	$\phi 1.1 \sqrt{f'_c}$	$\phi 4 \sqrt{f'_c}$
422.7.2.1	$\sqrt{f'_c} \leq 8.3$ in MPa	$\sqrt{f'_c} \leq 27$ in kgf/cm ²	$\sqrt{f'_c} \leq 100$ in psi
422.7.4.1(a)(a)	$T_{th} < 0.083 \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right)$	$T_{th} < 0.27 \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right)$	$T_{th} < \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right)$
422.7.4.1(a)(b)	$T_{th} < 0.083 \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{0.33 \lambda \sqrt{f'_c}}}$	$T_{th} < 0.27 \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{\sqrt{f'_c}}}$	$T_{th} < \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{4 \lambda \sqrt{f'_c}}}$
422.7.4.1(a)(c)	$T_{th} < 0.083 \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right) \sqrt{1 + \frac{N_u}{0.33 A_g \lambda \sqrt{f'_c}}}$	$T_{th} < 0.27 \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right) \sqrt{1 + \frac{N_u}{A_g \lambda \sqrt{f'_c}}}$	$T_{th} < \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right) \sqrt{1 + \frac{N_u}{4 A_g \lambda \sqrt{f'_c}}}$
422.7.4.1(b)(a)	$T_{th} < 0.083 \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right)$	$T_{th} < 0.27 \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right)$	$T_{th} < \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right)$
422.7.4.1(b)(b)	$T_{th} < 0.083 \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{0.33 \lambda \sqrt{f'_c}}}$	$T_{th} < 0.27 \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{\sqrt{f'_c}}}$	$T_{th} < \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{4 \lambda \sqrt{f'_c}}}$
422.7.4.1(b)(c)	$T_{th} < 0.083 \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right) \sqrt{1 + \frac{N_u}{0.33 A_g \lambda \sqrt{f'_c}}}$	$T_{th} < 0.27 \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right) \sqrt{1 + \frac{N_u}{A_g \lambda \sqrt{f'_c}}}$	$T_{th} < \lambda \sqrt{f'_c} \left(\frac{A_g^2}{P_{cp}} \right) \sqrt{1 + \frac{N_u}{4 A_g \lambda \sqrt{f'_c}}}$

422.7.5.1(a)	$T_{cr} = 0.33\lambda\sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right)$	$T_{cr} = \lambda\sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right)$	$T_{cr} = 4\lambda\sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right)$
422.7.5.1(b)	$T_{cr} = 0.33\lambda\sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{0.33\lambda\sqrt{f'_c}}}$	$T_{cr} = \lambda\sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{\lambda\sqrt{f'_c}}}$	$T_{cr} = 4\lambda\sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{4\lambda\sqrt{f'_c}}}$
422.7.5.1(c)	$T_{cr} = 0.33\lambda\sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) \sqrt{1 + \frac{N_u}{0.33A_g\lambda\sqrt{f'_c}}}$	$T_{cr} = \lambda\sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) \sqrt{1 + \frac{N_u}{A_g\lambda\sqrt{f'_c}}}$	$T_{cr} = 4\lambda\sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) \sqrt{1 + \frac{N_u}{4A_g\lambda\sqrt{f'_c}}}$
422.7.7.1a	$\sqrt{\left(\frac{V_u}{b_w d} \right)^2 + \left(\frac{T_u P_u}{1.7 A_{oh}^2} \right)^2} \leq \phi \left(\frac{V_c}{b_w d} + 0.66\sqrt{f'_c} \right)$	$\sqrt{\left(\frac{V_u}{b_w d} \right)^2 + \left(\frac{T_u P_u}{1.7 A_{oh}^2} \right)^2} \leq \phi \left(\frac{V_c}{b_w d} + 2\sqrt{f'_c} \right)$	$\sqrt{\left(\frac{V_u}{b_w d} \right)^2 + \left(\frac{T_u P_u}{1.7 A_{oh}^2} \right)^2} \leq \phi \left(\frac{V_c}{b_w d} + 8\sqrt{f'_c} \right)$
422.7.7.1b	$\left(\frac{V_u}{b_w d} \right)^2 + \left(\frac{T_u P_u}{1.7 A_{oh}^2} \right)^2 \leq \phi \left(\frac{V_c}{b_w d} + 0.66\sqrt{f'_c} \right)$	$\left(\frac{V_u}{b_w d} \right)^2 + \left(\frac{T_u P_u}{1.7 A_{oh}^2} \right)^2 \leq \phi \left(\frac{V_c}{b_w d} + 2\sqrt{f'_c} \right)$	$\left(\frac{V_u}{b_w d} \right)^2 + \left(\frac{T_u P_u}{1.7 A_{oh}^2} \right)^2 \leq \phi \left(\frac{V_c}{b_w d} + 8\sqrt{f'_c} \right)$
422.9.4.4(b), (c), and (e)	$(3.3 + 0.08f'_c)A_c$ $11A_c$ $5.5A_c$	$(34 + 0.08f'_c)A_c$ $110A_c$ $55A_c$	$(480 + 0.08f'_c)A_c$ $1600A_c$ $800A_c$
424.3.2	$s = 380 \left(\frac{280}{f_s} \right) - 2.5c_c$ $s = 300 \left(\frac{280}{f_s} \right)$	$s = 38 \left(\frac{2800}{f_s} \right) - 2.5c_c$ $s = 30 \left(\frac{2800}{f_s} \right)$	$s = 15 \left(\frac{40,000}{f_s} \right) - 2.5c_c$ $s = 12 \left(\frac{40,000}{f_s} \right)$
424.3.2.2	$\Delta f_{ps} \leq 250 \text{ MPa}$ $\Delta f_{ps} \leq 140 \text{ MPa}$	$\Delta f_{ps} \leq 2500 \text{ kgf/cm}^2$ $\Delta f_{ps} \leq 1400 \text{ kgf/cm}^2$	$\Delta f_{ps} \leq 36,000 \text{ psi}$ $\Delta f_{ps} \leq 20,000 \text{ psi}$
424.4.3.2	$\frac{0.0018 \times 420}{f_y}$	$\frac{0.0018 \times 4200}{f_y}$	$\frac{0.0018 \times 60,000}{f_y}$
424.5.2.1	$f_t \leq 0.62\sqrt{f'_c}$ $0.62\sqrt{f'_c} < f_t \leq 1.0\sqrt{f'_c}$ $f_t > 1.0\sqrt{f'_c}$ $f_t \leq 0.50\sqrt{f'_{ci}}$	$f_t \leq 2\sqrt{f'_c}$ $2\sqrt{f'_c} < f_t \leq 3.2\sqrt{f'_c}$ $f_t > 3.2\sqrt{f'_c}$ $f_t \leq 1.6\sqrt{f'_{ci}}$	$f_t \leq 7.5\sqrt{f'_c}$ $7.5\sqrt{f'_c} < f_t \leq 12\sqrt{f'_c}$ $f_t > 12\sqrt{f'_c}$ $f_t \leq 6\sqrt{f'_{ci}}$
424.5.3.1	$0.50\sqrt{f'_{ci}}$ $0.25\sqrt{f'_{ci}}$	$1.6\sqrt{f'_{ci}}$ $0.8\sqrt{f'_{ci}}$	$6\sqrt{f'_{ci}}$ $3\sqrt{f'_{ci}}$
424.4.1.4	$\sqrt{f'_c} \leq 8.3 \text{ MPa}$	$\sqrt{f'_c} \leq 26.5 \text{ kgf/cm}^2$	$\sqrt{f'_c} \leq 100 \text{ psi}$
425.4.2.2	$\ell_d = \left(\frac{f_y \psi_t \psi_e}{2.1\lambda\sqrt{f'_c}} \right) d_b$	$\ell_d = \left(\frac{f_y \psi_t \psi_e}{6.6\lambda\sqrt{f'_c}} \right) d_b$	$\ell_d = \left(\frac{f_y \psi_t \psi_e}{25\lambda\sqrt{f'_c}} \right) d_b$
425.4.2.3a	$\ell_d = \frac{f_y \psi_t \psi_e \psi_s}{1.1\lambda\sqrt{f'_c} \left(\frac{c_b + K_{tr}}{d_b} \right)} d_b$	$\ell_d = \frac{f_y \psi_t \psi_e \psi_s}{3.5\lambda\sqrt{f'_c} \left(\frac{c_b + K_{tr}}{d_b} \right)} d_b$	$\ell_d = \frac{3f_y \psi_t \psi_e \psi_s}{40\lambda\sqrt{f'_c} \left(\frac{c_b + K_{tr}}{d_b} \right)} d_b$
425.4.4.2(a)	$\left(\frac{0.19f_y \psi_e}{\sqrt{f'_c}} \right) d_b$	$\left(\frac{0.06f_y \psi_e}{\sqrt{f'_c}} \right) d_b$	$\left(\frac{0.016f_y \psi_e}{\sqrt{f'_c}} \right) d_b$
425.4.6.3(a)	$\left(\frac{f_y - 240}{f_y} \right)$	$\left(\frac{f_y - 2460}{f_y} \right)$	$\left(\frac{f_y - 35,000}{f_y} \right)$
425.4.6.3(b)	$3.3 \left(\frac{f_y}{\lambda\sqrt{f'_c}} \right) \left(\frac{A_b}{s} \right)$	$\left(\frac{f_y}{\lambda\sqrt{f'_c}} \right) \left(\frac{A_b}{s} \right)$	$0.27 \left(\frac{f_y}{\lambda\sqrt{f'_c}} \right) \left(\frac{A_b}{s} \right)$
425.4.8.1(a)	$\left(\frac{f_{se}}{21} \right) d_b + \left(\frac{f_{ps} + f_{se}}{7} \right) d_b$	$\left(\frac{f_{se}}{210} \right) d_b + \left(\frac{f_{ps} + f_{se}}{70} \right) d_b$	$\left(\frac{f_{se}}{3000} \right) d_b + \left(\frac{f_{ps} + f_{se}}{1000} \right) d_b$
425.4.9.2(a)	$\left(\frac{0.24f_y}{\lambda\sqrt{f'_c}} \right) d_b$	$\left(\frac{0.075f_y}{\lambda\sqrt{f'_c}} \right) d_b$	$\left(\frac{f_y}{50\lambda\sqrt{f'_c}} \right) d_b$
425.4.9.2(b)	$(0.043f_y) d_b$	$(0.0044f_y) d_b$	$(0.0003f_y) d_b$
425.5.1(a) and (b)	$0.071f_y d_b$ $(0.13f_y - 24) d_b$	$0.0073f_y d_b$ $(0.013f_y - 24) d_b$	$0.0005f_y d_b$ $(0.0009f_y - 24) d_b$
425.7.1.3(b)	$0.17 \frac{d_b f_{yt}}{\lambda\sqrt{f'_c}}$	$0.053 \frac{d_b f_{yt}}{\lambda\sqrt{f'_c}}$	$0.014 \frac{d_b f_{yt}}{\lambda\sqrt{f'_c}}$
425.7.1.7	$A_b f_{yt} \leq 40,000 \text{ N}$	$A_b f_{yt} \leq 4000 \text{ kgf}$	$A_b f_{yt} \leq 9000 \text{ lb}$
425.9.4.5.1	$f_{ps} = f_{se} + 70$	$f_{ps} = f_{se} + 700$	$f_{ps} = f_{se} + 10,000$
426.12.5.1	$0.62\sqrt{f'_c}$	$2\sqrt{f'_c}$	$7.5\sqrt{f'_c}$

Chapter 5

STRUCTURAL STEEL

NATIONAL STRUCTURAL CODE OF THE PHILIPPINES VOLUME I BUILDINGS, TOWERS AND OTHER VERTICAL STRUCTURES

SEVENTH EDITION, 2015

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Table of Contents

PART 1 - SPECIFICATION FOR STEEL MEMBERS.....	5-11
SYMBOLS.....	5-11
DEFINITIONS.....	5-19
SECTION 501 - GENERAL PROVISIONS.....	5-35
501.1 Scope.....	5-35
501.2 Referenced Specifications, Codes and Standards.....	5-35
501.3 Material.....	5-38
501.4 Structural Design Drawings and Specifications.....	5-40
SECTION 502 - DESIGN REQUIREMENTS.....	5-40
502.1 General Provisions.....	5-40
502.2 Loads and Load Combinations.....	5-40
502.3 Design Basis.....	5-40
502.4 Classification of Sections for Local Buckling.....	5-42
502.5 Fabrication, Erection and Quality Control.....	5-46
502.6 Evaluation of Existing Structures.....	5-46
SECTION 503 - STABILITY ANALYSIS AND DESIGN.....	5-47
503.1 Stability Design Requirements.....	5-47
503.2 Calculation of Required Strengths.....	5-48
SECTION 504 - DESIGN OF MEMBERS FOR TENSION.....	5-51
504.1 Slenderness Limitations.....	5-51
504.2 Tensile Strength.....	5-51
504.3 Area Determination.....	5-51
504.4 Built-up Members.....	5-52
504.5 Pin-Connected Members.....	5-52
504.6 Eyebars.....	5-52
SECTION 505 - DESIGN OF MEMBERS FOR COMPRESSION.....	5-55
505.1 General Provisions.....	5-55
505.2 Slenderness Limitations and Effective Length.....	5-55
505.3 Compressive Strength for Flexural Buckling of Members Without Slender Elements.....	5-55
505.4 Compressive Strength for Torsional and Flexural-Torsional Buckling of Members without Slender Elements.....	5-55
505.5 Single Angle Compression Members.....	5-56
505.6 Built-up Members.....	5-57
505.7 Members with Slender Elements.....	5-59
SECTION 506 - DESIGN OF MEMBERS FOR FLEXURE.....	5-62
506.1 General Provisions.....	5-62
506.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent about their Major Axis.....	5-64
506.3 Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about their Major Axis.....	5-65
506.4 Other I-Shaped Members with Compact or Noncompact Webs Bent about their Major Axis.....	5-65
506.5 Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about their Major Axis.....	5-67
506.6 I-Shaped Members and Channels Bent about their Minor Axis.....	5-68
506.7 Square and Rectangular HSS and Box-shaped Members.....	5-68
506.8 Round HSS.....	5-69
506.9 Tees and Double Angles Loaded in the Plane of Symmetry.....	5-69

506.10 Single Angles	5-70
506.11 Rectangular Bars and Rounds	5-71
506.12 Unsymmetrical Shapes	5-72
506.13 Proportions of Beams and Girders	5-72
SECTION 507 - DESIGN OF MEMBERS FOR SHEAR.....	5-74
507.1 General Provisions	5-74
507.2 Members with Unstiffened or Stiffened Webs	5-74
507.3 Tension Field Action	5-75
507.4 Single Angles	5-76
507.5 Rectangular HSS and Box Members	5-76
507.6 Round HSS	5-76
507.7 Weak Axis Shear in Singly and Doubly Symmetric Shapes	5-76
507.8 Beams and Girders with Web Openings	5-76
SECTION 508 - DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION	5-77
508.1 Doubly and Singly Symmetric Members Subject to Flexure and Axial Force	5-77
508.2 Unsymmetric and other Members Subject to Flexure and Axial Force	5-78
508.3 Members under Torsion and Combined Torsion, Flexure, Shear and/or Axial Force	5-79
SECTION 509 - DESIGN OF COMPOSITE MEMBERS.....	5-81
509.1 General Provisions	5-81
509.2 Axial Members	5-81
509.3 Flexural Members	5-84
509.4 Flexural Strength of Concrete-Encased and Filled Members	5-88
509.5 Combined Axial Force and Flexure	5-88
509.6 Special Cases	5-88
SECTION 510 - DESIGN OF CONNECTIONS.....	5-89
510.1 General Provisions	5-89
510.2 Welds	5-90
510.3 Bolts and Threaded Parts	5-98
510.4 Affected Elements of Members and Connecting Elements	5-102
510.5 Fillers	5-103
510.6 Splices	5-104
510.7 Bearing Strength	5-104
510.8 Column Bases and Bearing on Concrete	5-104
510.9 Anchor Rods and Embedments	5-104
510.10 Flanges and Webs with Concentrated Forces	5-105
SECTION 511 - DESIGN OF HSS AND BOX MEMBER CONNECTIONS.....	5-109
511.1 Concentrated Forces on HSS	5-109
511.2 HSS-to-HSS Truss Connections	5-111
511.3 HSS-to-HSS Moment Connections	5-118
SECTION 512 - DESIGN FOR SERVICEABILITY	5-122
512.1 General Provisions	5-122
512.2 Camber	5-122
512.3 Deflections	5-122
512.4 Drift	5-123
512.5 Vibration	5-123
512.6 Wind-Induced Motion	5-123
512.7 Expansion and Contraction	5-123
512.8 Connection Slip	5-123

SECTION 513 - FABRICATION, ERECTION AND QUALITY CONTROL	5-123
513.1 Shop and Erection Drawings	5-123
513.2 Fabrication	5-123
513.3 Shop Painting	5-125
513.4 Erection	5-125
513.5 Quality Control	5-126
APPENDIX A - INELASTIC ANALYSIS AND DESIGN.....	5-126
A-1.1 General Provisions	5-126
A-1.2 Materials	5-126
A-1.3 Moment Redistribution	5-126
A-1.4 Local Buckling.....	5-126
A-1.5 Stability and Second-Order Effects.....	5-127
A-1.6 Columns and Other Compression Members	5-127
A-1.7 Beams and Other Flexural Members.....	5-128
A-1.8 Members under Combined Forces	5-128
A-1.9 Connections.....	5-128
APPENDIX A-2 - DESIGN FOR PONDING.....	5-129
A-2.1 Simplified Design for Ponding.....	5-129
A-2.2 Improved Design for Ponding	5-129
APPENDIX A-3 - DESIGN FOR FATIGUE	5-130
A-3.1 General Provisions	5-130
A-3.2 Calculation of Maximum Stresses and Stress Ranges.....	5-130
A-3.3 Design Stress Range.....	5-131
A-3.4 Bolts and Threaded Parts	5-132
A-3.5 Special Fabrication and Erection Requirements	5-132
APPENDIX A-4 - STRUCTURAL DESIGN FOR FIRE CONDITIONS.....	5-147
A-4.1 General Provisions	5-147
A-4.2 Structural Design for Fire Conditions by Analysis	5-147
A-4.3 Design by Qualification Testing	5-150
APPENDIX A-5 - EVALUATION OF EXISTING STRUCTURES.....	5-151
A-5.1 General Provisions	5-151
A-5.2 Material Properties.....	5-151
A-5.3 Evaluation by Structural Analysis.....	5-151
A-5.4 Evaluation by Load Tests.....	5-152
A-5.5 Evaluation Report	5-152
APPENDIX A-6 - STABILITY BRACING FOR COLUMNS AND BEAMS	5-153
A-6.1 General Provisions	5-153
A-6.2 Columns	5-153
A-6.3 Beams.....	5-153
APPENDIX A-7 - DIRECT ANALYSIS METHOD	5-156
A-7.1 General Requirements.....	5-156
A-7.2 Notional Loads.....	5-156
A-7.3 Design-Analysis Constraints.....	5-156

PART 2A - SEISMIC PROVISION FOR STRUCTURAL STEEL BUILDINGS	5-157
SYMBOLS.....	5-157
PART 2B	
SECTION 514 - STRUCTURAL STEEL BUILDING PROVISIONS	5-160
514.1 Scope.....	5-160
SECTION 515 - REFERENCED SPECIFICATIONS, CODES, AND STANDARDS.....	5-161
SECTION 516 - GENERAL SEISMIC DESIGN REQUIREMENTS.....	5-161
SECTION 517 - LOADS, LOAD COMBINATIONS, AND NOMINAL STRENGTHS.....	5-162
517.1 Loads and Load Combinations.....	5-162
517.2 Nominal Strength	5-162
SECTION 518 - STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS, SHOP DRAWINGS, AND ERECTION DRAWINGS.....	5-162
518.1 Structural Design Drawings and Specifications	5-162
518.2 Shop Drawings	5-163
518.3 Erection Drawings.....	5-163
SECTION 519 - MATERIALS	5-163
519.1 Material Specifications.....	5-163
519.2 Material Properties for Determination of Required Strength of Members and Connections.....	5-163
519.3 Heavy Section CVN Requirements	5-164
SECTION 520 - CONNECTIONS, JOINTS, AND FASTENERS	5-165
520.1 Scope.....	5-165
520.2 Bolted Joints.....	5-165
520.3 Welded Joints	5-165
520.4 Protected Zone.....	5-166
520.5 Continuity Plates and Stiffeners	5-166
SECTION 521 - MEMBERS.....	5-167
521.1 Scope.....	5-167
521.2 Classification of Sections for Local Buckling.....	5-167
521.3 Column Strength	5-169
521.4 Column Splices	5-169
521.5 Column Bases.....	5-169
521.6 H-Piles.....	5-170
SECTION A.- MOMENT FRAME SYSTEMS.....	5-171
SECTION A.3 - SPECIAL MOMENT FRAMES (SMF).....	5-171
522.1 Scope.....	5-171
522.2 Beam-to-Column Connections	5-171
522.3 Panel Zone of Beam-to-Column Connections (Beam Web Parallel to Column Web).....	5-172
522.4 Beam and Column Limitations.....	5-172
522.5 Continuity Plates	5-172
522.6 Column-Beam Moment Ratio	5-173
522.7 Lateral Bracing at Beam-to-Column Connections	5-174
522.8 Lateral Bracing of Beams.....	5-174
522.9 Column Splices	5-174

528.9 Protected Zone.....	5-187
528.10 Demand Critical Welds	5-187
SECTION B.4 - BUCKLING-RESTRAINED BRACED FRAMES (BRBF).....	5-187
529.1 Scope	5-187
529.2 Bracing Members	5-187
529.3 Bracing Connections	5-188
529.4 Special Requirements	5-189
529.5 Beams and Columns	5-189
529.6 Protected Zone.....	5-189
SECTION B.5 - SPECIAL PLATE SHEAR WALLS (SPSW).....	5-190
530.1 Scope	5-190
530.2 Webs.....	5-190
530.3 Connections of Webs to Boundary Elements	5-190
530.4 Horizontal and Vertical Boundary Elements	5-190
SECTION C. – COMPOSITE MOMENT-FRAME SYSTEMS	5-171
SECTION C.1 – COMPOSITE ORDINARY BRACED FRAMES (C-OBF).....	5-171
SECTION C.2 – COMPOSITE SPECIAL CONCENTRICALLY BRACED FRAMES (C-SCBF).....	5-171
SECTION C.3 – COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF)	5-171
SECTION C.4 – COMPOSITE ORDINARY SHEAR WALLS (C-OSW)	5-171
SECTION C.5 – COMPOSITE SPECIAL SHEAR WALLS (C-SSW).....	5-171
SECTION C.6 – COMPOSITE PLATE SHEAR WALLS (C-PSW).....	5-171
SECTION 531 - QUALITY ASSURANCE PLAN.....	5-191
531.1 Scope	5-191
B-1. PREQUALIFICATION OF BEAM-COLUMN AND LINK-TO-COLUMN CONNECTIONS.....	5-192
B-1.1 Scope	5-192
B-1.2 General Requirements	5-192
B-1.3 Testing Requirements	5-192
B-1.4 Prequalification Variables	5-192
B-1.5 Design Procedure	5-193
B-1.6 Prequalification Record	5-193
B-2. QUALITY ASSURANCE PLAN.....	5-194
B-2.1 Scope	5-194
B-2.2 Inspection and Nondestructive Testing Personnel.....	5-194
B-2.3 Contractor Documents.....	5-194
B-2.4 Quality Assurance Agency Documents	5-194
B-2.5 Inspection Points and Frequencies	5-195
B-3. SEISMIC DESIGN COEFFICIENTS AND APPROXIMATE PERIOD PARAMETERS.....	5-199
B-3.1 Scope	5-199
B-3.2 Symbols	5-199
B-4. QUALIFYING CYCLIC TESTS OF BEAM-TO-COLUMN AND LINK-TO-COLUMN CONNECTIONS...5-200	
B-4.1 Scope	5-200
B-4.2 Symbols.....	5-200
B-4.3 Definitions	5-200
B-4.4 Test Subassemblage Requirements	5-200
B-4.5 Essential Test Variables	5-201
B-4.6 Loading History.....	5-202

B-4.7 Instrumentation	5-203
B-4.8 Materials Testing Requirements	5-203
B-4.9 Test Reporting Requirements	5-203
B-4.10 Acceptance Criteria	5-204
B-5. QUALIFYING CYCLIC TESTS OF BUCKLING-RESTRAINED BRACES	5-205
B-5.1 Scope	5-205
B-5.2 Symbols	5-205
B-5.3 Definitions	5-205
B-5.4 Subassemblage Test Specimen	5-205
B-5.5 Brace Test Specimen	5-206
B-5.6 Loading History	5-206
B-5.7 Instrumentation	5-207
B-5.8 Materials Testing Requirements	5-207
B-5.9 Test Reporting Requirements	5-207
B-5.10 Acceptance Criteria	5-208
B-6. WELDING PROVISIONS	5-208
B-6.1 Scope	5-208
B-6.2 Structural Design Drawings and Specifications, Shop Drawings, and Erection Drawings	5-208
B-6.3 Personnel	5-209
B-6.4 Nondestructive Testing Procedures	5-209
B-6.5 Additional Welding Provisions	5-210
B-6.6 Additional Welding Provisions for Demand Critical Welds Only	5-210
B-7 WELD METAL/WELDING PROCEDURE SPECIFICATION NOTCH TOUGHNESS VERIFICATION TEST	5-211
B-7.1 Scope	5-211
B-7.2 Test Conditions	5-212
B-7.3 Test Specimens	5-212
B-7.4 Acceptance Criteria	5-212
PART 2B - COMPOSITE STRUCTURAL STEEL AND REINFORCED CONCRETE BUILDINGS	5-213
SECTION 532 - SCOPE	5-213
SECTION 533 - REFERENCED SPECIFICATIONS, CODES, AND STANDARDS	5-213
SECTION 534 - GENERAL SEISMIC DESIGN REQUIREMENTS	5-214
SECTION 535 - LOADS, LOAD COMBINATIONS, AND NOMINAL STRENGTHS	5-214
535.1 Loads and Load Combinations	5-214
535.2 Nominal Strength	5-214
SECTION 536 - MATERIALS	5-215
536.1 Structural Steel	5-215
536.2 Concrete and Steel Reinforcement	5-215
SECTION 537 - COMPOSITE MEMBERS	5-215
537.1 Scope	5-215
537.2 Composite Floor and Roof Slabs	5-215
537.3 Composite Beams	5-215
537.4 Encased Composite Columns	5-215
537.5 Filled Composite Columns	5-218
SECTION 538 - COMPOSITE CONNECTIONS	5-218
538.1 Scope	5-218
538.2 General Requirements	5-218

538.3	Nominal Strength of Connections	5-218
SECTION 539 - COMPOSITE PARTIALLY RESTRAINED (PR) MOMENT FRAMES (C-PRMF).....		5-220
539.1	Scope	5-220
539.2	Columns	5-220
539.3	Composite Beams.....	5-220
539.4	Moment Connections	5-220
SECTION 540 - COMPOSITE SPECIAL MOMENT FRAMES (C-SMF).....		5-220
540.1	Scope.....	5-220
540.2	Columns	5-220
540.3	Beams	5-220
540.4	Moment Connections	5-221
540.5	Column-Beam Moment Ratio	5-221
SECTION 541 - COMPOSITE INTERMEDIATE MOMENT FRAMES (C-IMF)		5-221
541.1	Scope	5-221
541.2	Columns	5-221
541.3	Beams	5-221
541.4	Moment Connections	5-221
SECTION 542 - COMPOSITE ORDINARY MOMENT FRAMES (C-OMF)		5-222
542.1	Scope.....	5-222
542.2	Columns	5-222
542.3	Beams	5-222
542.4	Moment Connections	5-222
SECTION 543 - COMPOSITE SPECIAL CONCENTRICALLY BRACED FRAMES (C-CBF)		5-222
543.1	Scope	5-222
543.2	Columns	5-222
543.3	Beams	5-222
543.4	Braces.....	5-222
543.5	Connections.....	5-222
SECTION 544 - COMPOSITE ORDINARY BRACED FRAMES (C-OBF).....		5-223
544.1	Scope	5-223
544.2	Columns	5-223
544.3	Beams	5-223
544.4	Braces	5-223
544.5	Connections	5-223
SECTION 545 - COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF).....		5-223
545.1	Scope	5-223
545.2	Columns	5-223
545.3	Links.....	5-223
545.4	Braces	5-224
545.5	Connections	5-224
SECTION 546 - ORDINARY REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS (C-ORCW).....		5-224
546.1	Scope	5-224
546.2	Boundary Members	5-224
546.3	Steel Coupling Beams	5-225
546.4	Encased Composite Coupling Beams.....	5-225

SECTION 547 - SPECIAL REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS (C-SRCW)	5-225
547.1 Scope	5-225
547.2 Boundary Members	5-225
547.3 Steel Coupling Beams	5-226
547.4 Encased Composite Coupling Beams	5-226
SECTION 548 - COMPOSITE STEEL PLATE SHEAR WALLS (C-SPW)	5-226
548.1 Scope	5-226
548.2 Wall Elements	5-226
548.3 Boundary Members	5-227
548.4 Openings	5-227
SECTION 549 - STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS, SHOP DRAWINGS, AND ERECTION DRAWINGS	5-227
SECTION 550 - QUALITY ASSURANCE PLAN	5-228
PART 3 - DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS	5-228
SYMBOLS	5-228
SECTION 551 - GENERAL PROVISIONS	5-235
551.1 Scope, Applicability and Definitions	5-235
551.2 Material	5-236
551.3 Loads	5-238
551.4 Allowable Strength Design	5-238
551.5 Load and Resistance Factor Design	5-239
551.7 Yield Stress and Strength Increase from Cold Work of Forming	5-239
551.8 Serviceability	5-240
551.9 Referenced Documents	5-240
SECTION 552 - ELEMENTS	5-242
552.1 Dimensional Limits and Considerations	5-242
552.2 Effective Widths of Stiffened Elements	5-243
552.3 Effective Widths of Unstiffened Elements	5-248
552.4 Effective Width of Uniformly Compressed Elements with a Simple Lip Edge Stiffener	5-251
552.5 Effective Widths of Stiffened Elements with Single or Multiple Intermediate Stiffeners or Edge Stiffened Elements with Intermediate Stiffener(s)	5-253
SECTION 553 - MEMBERS	5-255
553.1 Properties of Sections	5-255
553.2 Tension Members	5-255
553.3 Flexural Members	5-255
553.4 Concentrically Loaded Compression Members	5-273
553.5 Combined Axial Load and Bending	5-277
SECTION 554 - STRUCTURAL ASSEMBLIES AND SYSTEMS	5-280
554.1 Built-Up Sections	5-280
554.2 Mixed Systems	5-282
554.3 Lateral and Stability Bracing	5-282
554.4 Cold-Formed Steel Light-Frame Construction	5-284
554.5 Floor, Roof, or Wall Steel Diaphragm Construction	5-284
554.6 Metal Roof and Wall System	5-285

SECTION 555 - CONNECTIONS AND JOINTS.....	5-291
555.1 General Provisions	5-291
555.2 Welded Connections.....	5-291
555.3 Bolted Connection.....	5-299
555.4 Screw Connections.....	5-301
555.5 Rupture.....	5-304
555.5 Connecting to Other Materials	5-304
SECTION 556 - TESTS FOR SPECIAL CASES	5-304
556.1 Tests for Determining Structural Performance.....	5-304
556.2 Tests for Confirming Structural Performance	5-307
556.3 Tests for Determining Mechanical Properties	5-307
SECTION 557 - DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS AND CONNECTIONS FOR CYCLIC LOADING (FATIGUE).....	5-308
557.1 General	5-308
557.2 Calculation of Maximum Stresses and Stress Ranges	5-310
557.3 Design Stress Range	5-310
557.4 Bolts and Threaded Parts.....	5-310
557.5 Special Fabrication Requirements	5-311
SECTION C-1 - DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS USING THE DIRECT STRENGTH METHOD	5-311
C-1 Design of Cold-Formed Steel Structural Members Using the Direct Strength Method	5-311
SECTION C-2 - SECOND-ORDER ANALYSIS.....	5-318
C.2.1 General Requirements	5-318
C.2.2 Design and Analysis Constraints.....	5-318
SECTION C3 - ADDITIONAL PROVISIONS.....	5-319
C.3.1 Scope.....	5-319
C.3.2 Other Steels	5-319
C.3.3 Loads	5-319
C.3.4 Referenced Documents.....	5-319
C.3.5 Tension Members	5-320
C.3.6 Light-Frame Steel Construction	5-320
C.3.7 Welded Connections.....	5-321
C.3.8 Bolted Connections	5-322
C.3.9 Rupture	5-327

This Chapter includes the following:

Part 1 Specification for Steel Members

Appendix A

Part 2A Seismic Provisions for Structural Steel Buildings

Appendix B

Part 2B Seismic Provisions for Composite Structural Steel and Reinforced Concrete Buildings

Part 3 Specifications for Design of Cold-Formed Steel Structural Members

Appendix C

PART 1 SPECIFICATION FOR STEEL MEMBERS

SYMBOLS

A	=	Column cross-sectional area, mm^2
	=	Total cross-sectional area of member, mm^2
A_B	=	Loaded area of concrete, mm^2
A_{BM}	=	Cross-sectional area of the base metal, mm^2
A_b	=	Nominal unthreaded body area of bolt or threaded part, mm^2
A_{bi}	=	Cross-sectional area of the overlapping branch, mm^2
A_{bj}	=	Cross-sectional area of the overlapped branch, mm^2
A_c	=	Area of concrete, mm^2
A_c	=	Area of concrete slab within effective width, mm^2
A_D	=	Area of an upset rod based on the major thread diameter, mm^2
A_e	=	Effective net area, mm^2
A_{eff}	=	Summation of the effective areas of the cross section based on the reduced effective width, b_e , mm^2
A_{fc}	=	Area of compression flange, mm^2
A_{fg}	=	Gross tension flange area, mm^2
A_{fn}	=	Net tension flange area, mm^2
A_{ft}	=	Area of tension flange, mm^2
A_g	=	Gross area of member, mm^2
A_g	=	Gross area of section based on design wall thickness, mm^2
A_g	=	Gross area of composite member, mm^2
A_g	=	Chord gross area, mm^2
A_{gv}	=	Gross area subject to shear, mm^2
A_n	=	Net area of member, mm^2
A_{nt}	=	Net area subject to tension, mm^2
A_{nv}	=	Net area subject to shear, mm^2
A_{pb}	=	Projected bearing area, mm^2
A_r	=	Area of adequately developed longitudinal reinforcing steel within the effective width of the concrete slab, mm^2
A_s	=	Area of steel cross section, mm^2
A_{sc}	=	Cross-sectional area of stud shear connector, mm^2
A_{sf}	=	Shear area on the failure path, mm^2
A_{sr}	=	Area of continuous reinforcing bars, mm^2
A_{st}	=	Stiffener area, mm^2
A_t	=	Net tensile area, mm^2

A_w	=	Web area, the overall depth times the web thickness, dt_w , mm ²	D	=	Outside diameter, mm
A_w	=	Effective area of the weld, mm ²	D	=	Outside diameter of round HSS main member, mm
A_{wi}	=	Effective area of weld throat of any <i>ith</i> weld element, mm ²	D	=	Chord diameter, mm
A_1	=	Area of steel concentrically bearing on a concrete support, mm ²	D_b	=	Outside diameter of round HSS branch member, mm
A_2	=	Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm ²	D_s	=	Factor used in Eq. 507.3-3, dependent on the type of transverse stiffeners used in a plate girder
B	=	Overall width of rectangular hollow structural sections (HSS) member, measured 90° to the plane of the connection, mm	D_u	=	In slip-critical connections, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension
B	=	Overall width of rectangular HSS main member, measured 90° to the plane of the connection, mm	E, E_s	=	Modulus of elasticity of steel 200,000 MPa
B	=	Factor for lateral-torsional buckling in tees and double angles	E_c	=	Modulus of elasticity of concrete
B_b	=	Overall width of rectangular HSS branch member, measured 90° to the plane of the connection, mm	E_{cm}	=	Modulus of elasticity of concrete at elevated temperature, MPa
B_{bi}	=	Overall branch width of the overlapping branch, mm	EI_{eff}	=	Effective stiffness of composite section, N-mm ²
B_{bj}	=	Overall branch width of the overlapped branch, mm	E_m	=	Modulus of elasticity of steel at elevated temperature, MPa
B_p	=	Width of plate, transverse to the axis of the main member, mm	F_a	=	Available axial stress at the point of consideration, MPa
	=	Width of plate, measure 90° to the plane of connection, mm	F_{BM}	=	Nominal strength of the base metal per unit area, MPa
B_1, B_2	=	Factors used in determining M_u for combined bending and axial forces when first-order analysis is employed	F_{bw}	=	Available flexural stress at the point of consideration about the major axis, MPa
C	=	HSS torsional constant	F_{bz}	=	Available flexural stress at the point of consideration about the minor axis, MPa
C_b	=	Lateral-torsional buckling modification factor for non-uniform moment diagrams when both ends of the unsupported segment are braced	F_c	=	Available stress, MPa
C_d	=	Coefficient relating relative brace stiffness and curvature	F_{cr}	=	Critical stress, MPa
C_f	=	Constant based on stress category, given in Table 501-3.1	F_{cr}	=	Buckling stress for the section as determined by analysis, MPa
C_m	=	Coefficient assuming no lateral translation of the frame	F_{cry}	=	Critical stress about the minor axis, MPa
C_p	=	Ponding flexibility coefficient for primary member in a flat roof	F_{crz}	=	Critical torsional buckling stress, MPa
C_r	=	Coefficient for web sidesway buckling	F_e	=	Elastic critical buckling stress, MPa
C_s	=	Ponding flexibility coefficient for secondary member in a flat roof	F_{ex}	=	Elastic flexural buckling stress about the major axis, MPa
C_v	=	Web shear coefficient	F_{EXX}	=	Electrode classification number, MPa
C_w	=	Warping constant, mm ⁶	F_{ey}	=	Elastic flexural buckling stress about the minor axis, MPa
D	=	Nominal dead load	F_{ez}	=	Elastic torsional buckling stress, MPa
D	=	Outside diameter of round HSS member, mm	F_L	=	A calculated stress used in the calculation of nominal flexural strength, MPa
			F_n	=	Nominal torsional strength
			F_n	=	Nominal tensile stress F_{nt} , or shear stress, F_{nv} , from Table 510.3.2, MPa
			F_{nt}	=	Nominal tensile stress from Table 510.3.2, MPa
			F'_n	=	Nominal tensile stress modified to include the effects of shearing stress, MPa
			F_{nv}	=	Nominal shear stress from Table 510.3.2, MPa
			F_{SR}	=	Design stress range, MPa

F_{TH}	=	Threshold fatigue stress range, maximum stress range for indefinite design life from Table A-3.1, MPa	G	=	Shear modulus of elasticity of steel 77,200 MPa
F_u	=	Specified minimum tensile strength of the type of steel being used, MPa	ΣH	=	Story shear produced by the lateral forces used to compute Δ_H , N
F_u	=	Specified minimum tensile strength of a stud shear connector, MPa	H	=	Overall height of rectangular HSS member, measured in the plane of the connection, mm
F_u	=	Specified minimum tensile strength of the connected material, MPa	H	=	Overall height of rectangular HSS main member, measured in the plane of the connection, mm
F_u	=	Specified minimum tensile strength of HSS material, MPa	H	=	Flexural constant
F_{um}	=	Specified minimum tensile strength of the type of steel being used at elevated temperature, MPa	H_b	=	Overall height of rectangular HSS branch member, measured in the plane of the connection, mm
F_w	=	Nominal strength of the weld metal per unit area, MPa	H_{bi}	=	Overall depth of the overlapping branch
F_{wi}	=	Nominal stress in any <i>ith</i> weld element, MPa	I	=	Moment of inertia in the place of bending, mm ⁴
F_{wix}	=	<i>x</i> component of stress F_{wi} , MPa	I	=	Moment of inertia about the axis of bending, mm ⁴
F_{wiy}	=	<i>y</i> component of stress F_{wi} , MPa	I_c	=	Moment of inertia of the concrete section, mm ⁴
F_y	=	Specified minimum yield stress of the type of steel being used, MPa. As used in this Specification, "yield stress" denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point)	I_d	=	Moment of inertia of the steel deck supported on secondary members, mm ⁴
F_y	=	Specified minimum yield stress of the compression flange, MPa	I_p	=	Moment of inertia of primary members, mm ⁴
F_y	=	Specified minimum yield stress of the column web, MPa	I_s	=	Moment of inertia of secondary members, mm ⁴
F_y	=	Specified minimum yield stress of HSS member material, MPa	I_s	=	Moment of inertia of steel shape, mm ⁴
F_y	=	Specified minimum yield stress of HSS main member material, MPa	I_{sr}	=	Moment of inertia of reinforcing bars, mm ⁴
F_{yb}	=	Specified minimum yield stress of HSS branch member material, MPa	I_x, I_y	=	Moment of inertia about the principal axes, mm ⁴
F_{ybi}	=	Specified minimum yield stress of the overlapping branch material, MPa	I_y	=	Out-of-plane moment of inertia, mm ⁴
F_{ybj}	=	Specified minimum yield stress of the overlapped branch material, MPa	I_z	=	Minor principal axis moment of inertia, mm ⁴
F_{yf}	=	Specified minimum yield stress of the flange, MPa	I_{yc}	=	Moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending referred to smaller flange, mm ⁴
F_{ym}	=	Specified minimum yield stress of the type of steel being used at elevated temperature, MPa	J	=	Torsional constant, mm ⁴
F_{yp}	=	Specified minimum yield stress of plate, MPa	K	=	Effective length factor determined in accordance with Section 503
F_{yr}	=	Specified minimum yield stress of reinforcing bars, MPa	K_z	=	Effective length factor for torsional buckling
F_{yst}	=	Specified minimum yield stress of the stiffener material, MPa	K_1	=	Effective length factor in the plane of bending, calculated based on the assumption of no lateral translation set equal to 1.0 unless analysis indicates that a smaller value may be used
F_{yw}	=	Specified minimum yield stress of the web, MPa	K_2	=	Effective length factor in the plane of bending, calculated based on a sidesway buckling analysis
			L	=	Story height, mm
			L	=	Length of the member, mm
			L	=	Actual length of end-loaded weld, mm
			L	=	Nominal occupancy live load

L	= Laterally unbraced length of a member, mm
L	= Span length, mm
L	= Length of member between work points at truss chord centerlines, mm
L_b	= Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm
L_b	= Distance between braces, mm
L_c	= Length of channel shear connector, mm
L_c	= Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, mm
L_e	= Total effective weld length of groove and fillet welds to rectangular HSS, mm
L_p	= Limiting laterally unbraced length for the limit state of yielding, mm
L_p	= Column spacing in direction of girder, m
L_{pd}	= Limiting laterally unbraced length for plastic analysis, mm
L_q	= Maximum unbraced length for M_r (the required flexural strength), mm
L_r	= Limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, mm
L_s	= Column spacing perpendicular to direction of girder, m
L_v	= Distance from maximum to zero shear force, mm
M_A	= Absolute value of moment at quarter point of the unbraced segment, N-mm
M_a	= Required flexural strength in chord, using ASD load combinations, N-mm
M_B	= Absolute value of moment at centerline of the unbraced segment, N-mm
M_{br}	= Required bracing moment, N-mm
M_C	= Absolute value of moment at three-quarter point of the unbraced segment, N-mm
$M_{c(x,y)}$	= Available flexural strength determined in accordance with Section 506, N-mm
M_{cx}	= Available flexural-torsional strength for strong axis flexure determined in accordance with Section 506, N-mm
M_e	= Elastic lateral-torsional buckling moment, N-mm
M_{lt}	= First-order moment under LRFD or ASD load combinations caused by lateral translation of the frame only, N-mm
M_{max}	= Absolute value of maximum moment in the unbraced segment, N-mm
M_n	= Nominal flexural strength, N-mm

M_{nt}	= First-order moment using LRFD or ASD load combinations assuming there is no lateral translation of the frame, N-mm
M_p	= Plastic bending moment, N-mm
M_r	= Required second-order flexural strength under LRFD or ASD load combinations, N-mm
M_r	= Required flexural strength using LRFD or ASD load combinations, N-mm
M_r	= Required flexural strength in chord, N-mm
M_{r-ip}	= Required in-plane flexural strength in branch, N-mm
M_{r-op}	= Required out-of-plane flexural strength in branch, N-mm
M_u	= Required flexural strength in chord using LRFD load combinations, N-mm
M_y	= Yield moment about the axis of bending, N-mm
M_1	= Smaller moment, calculated from a first-order analysis, at the ends of that portion of the member unbraced in the plane of bending under consideration, N-mm
M_2	= Larger moment, calculated from a first-order analysis, at the ends of that portion of the member unbraced in the plane of bending under consideration, N-mm
N	= Length of bearing (not less than k for end beam reactions), mm
N	= Bearing length of the load, measured parallel to the axis of the HSS member, (or measured across the width of the HSS in the case of the loaded cap plates), mm
N	= Number of stress range fluctuations in design life
N_b	= Number of bolts carrying the applied tension
N_i	= Additional lateral load
N_i	= Notional lateral load applied at level i , N
N_s	= Number of slip planes
O_v	= Overlap connection coefficient
P	= Pitch per thread, mm
P_{br}	= Required brace strength, N
P_c	= Available axial compressive strength, N
P_c	= Available tensile strength, N
P_{co}	= Available compressive strength out of the plane of bending, N
P_{e1}, P_{e2}	= Elastic critical buckling load for braced and unbraced frame, respectively, N
P_{eL}	= Euler buckling load, evaluated in the plane of bending, N
$P_{l(t,c)}$	= First-order axial force using LRFD or ASD load combinations as a result of lateral translation of the frame only (tension or compression), N

$P_{n(t,c)}$	=	First-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame (tension or compression), N	R_{wt}	=	Total nominal strength of transversely loaded fillet welds, as determined in accordance with Table 510.2.5 without the alternate in Section 510.2.4 (a)
P_n	=	Nominal axial strength, N	S	=	Elastic section modulus of round HSS, mm^3
P_o	=	Nominal axial compressive strength without consideration of length effects, N	S	=	Lowest elastic section modulus relative to the axis of bending, mm^3
P_p	=	Nominal bearing strength of concrete, N	S	=	Spacing of secondary members, m
P_r	=	Required second-order axial strength using LRFD or ASD load combinations, N	S	=	Chord elastic section modulus, mm^3
P_r	=	Required axial compressive strength using LRFD or ASD load combinations, N	S_c	=	Elastic section modulus to the toe in compression relative to the axis of bending, mm^3
P_r	=	Required strength, N	S_{eff}	=	Effective section modulus about major axis, mm^3
P_r	=	Required tensile strength using LRFD or ASD load combinations, N	S_{xt}, S_{xc}	=	Elastic section modulus referred to tension and compression flanges, respectively, mm^3
P_r	=	Required axial strength in branch, N	S_x, S_y	=	Elastic section modulus taken about the principal axes, mm^3
P_r	=	Required axial strength in chord, N	S_y	=	For channels, taken as the minimum section modulus
P_u	=	Required axial strength in compression, N	T	=	Nominal forces and deformations due to the design-basis fire defined in Section A-4.2.1
P_y	=	Member yield strength, N	T_a	=	Tension force due to ASD load combinations, kN
Q	=	Full reduction factor for slender compression elements	T_b	=	Minimum fastener tension given in Table 510.3.1, kN
Q_a	=	Reduction factor for slender stiffened compression elements	T_c	=	Available torsional strength, N-mm
Q_f	=	Chord-stress interaction parameter	T_n	=	Nominal torsional strength, N-mm
Q_n	=	Nominal strength of one stud shear connector, N	T_r	=	Required torsional strength, N-mm
Q_s	=	Reduction factor for slender unstiffened compression elements	T_u	=	Tension force due to LRFD load combinations, kN
R	=	Nominal load due to rainwater exclusive of the ponding contribution, MPa	U	=	Shear lag factor
R	=	Seismic response modification coefficient	U	=	Utilization ratio
R_a	=	Required strength (ASD)	U_{bs}	=	Reduction coefficient, used in calculating block shear rupture strength
R_{FIL}	=	Reduction factor for joints using a pair of transverse fillet welds only	U_p	=	Stress index
R_g	=	Coefficient to account for group effect	U_s	=	Stress index
R_m	=	Factor in Eq. 503.2-6b dependent on type of system	V	=	Required shear force introduced to column, N
R_m	=	Cross-section monosymmetry parameter	V'	=	Required shear force transferred by shear connectors, N
R_n	=	Nominal strength, specified in Section 502 through 511	V_c	=	Available shear strength, N
R_n	=	Nominal slip resistance, N	V_n	=	Nominal shear strength, N
R_p	=	Position effect factor for shear studs	V_r	=	Required shear strength at the location of the stiffener, N
R_{pc}	=	Web plastification factor	V_r	=	Required shear strength using LRFD or ASD load combinations, N
R_{PJP}	=	Reduction factor for reinforced or non-reinforced transverse partial-joint-penetration (PJP) groove welds	Y_i	=	Gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i , N
R_{pt}	=	Web plastification factor corresponding to the tension flange yielding limit state	Y_t	=	Hole reduction coefficient, N
R_u	=	Required strength (LRFD)			
R_{wt}	=	Total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table 510.2.5			

$P_{n(t,c)}$	=	First-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame (tension or compression), N	R_{wt}	=	Total nominal strength of transversely loaded fillet welds, as determined in accordance with Table 510.2.5 without the alternate in Section 510.2.4 (a)
P_n	=	Nominal axial strength, N	S	=	Elastic section modulus of round HSS, mm^3
P_o	=	Nominal axial compressive strength without consideration of length effects, N	S	=	Lowest elastic section modulus relative to the axis of bending, mm^3
P_p	=	Nominal bearing strength of concrete, N	S	=	Spacing of secondary members, m
P_r	=	Required second-order axial strength using LRFD or ASD load combinations, N	S	=	Chord elastic section modulus, mm^3
P_r	=	Required axial compressive strength using LRFD or ASD load combinations, N	S_c	=	Elastic section modulus to the toe in compression relative to the axis of bending, mm^3
P_r	=	Required strength, N	S_{eff}	=	Effective section modulus about major axis, mm^3
P_r	=	Required tensile strength using LRFD or ASD load combinations, N	S_{xt}, S_{xc}	=	Elastic section modulus referred to tension and compression flanges, respectively, mm^3
P_r	=	Required axial strength in branch, N	S_x, S_y	=	Elastic section modulus taken about the principal axes, mm^3
P_r	=	Required axial strength in chord, N	S_y	=	For channels, taken as the minimum section modulus
P_u	=	Required axial strength in compression, N	T	=	Nominal forces and deformations due to the design-basis fire defined in Section A-4.2.1
P_y	=	Member yield strength, N	T_a	=	Tension force due to ASD load combinations, kN
Q	=	Full reduction factor for slender compression elements	T_b	=	Minimum fastener tension given in Table 510.3.1, kN
Q_a	=	Reduction factor for slender stiffened compression elements	T_c	=	Available torsional strength, N-mm
Q_f	=	Chord-stress interaction parameter	T_n	=	Nominal torsional strength, N-mm
Q_n	=	Nominal strength of one stud shear connector, N	T_r	=	Required torsional strength, N-mm
Q_s	=	Reduction factor for slender unstiffened compression elements	T_u	=	Tension force due to LRFD load combinations, kN
R	=	Nominal load due to rainwater exclusive of the ponding contribution, MPa	U	=	Shear lag factor
R	=	Seismic response modification coefficient	U	=	Utilization ratio
R_a	=	Required strength (ASD)	U_{bs}	=	Reduction coefficient, used in calculating block shear rupture strength
R_{FIL}	=	Reduction factor for joints using a pair of transverse fillet welds only	U_p	=	Stress index
R_g	=	Coefficient to account for group effect	U_s	=	Stress index
R_m	=	Factor in Eq. 503.2-6b dependent on type of system	V	=	Required shear force introduced to column, N
R_m	=	Cross-section monosymmetry parameter	V'	=	Required shear force transferred by shear connectors, N
R_n	=	Nominal strength, specified in Section 502 through 511	V_c	=	Available shear strength, N
R_n	=	Nominal slip resistance, N	V_n	=	Nominal shear strength, N
R_p	=	Position effect factor for shear studs	V_r	=	Required shear strength at the location of the stiffener, N
R_{pc}	=	Web plastification factor	V_r	=	Required shear strength using LRFD or ASD load combinations, N
R_{PJP}	=	Reduction factor for reinforced or non-reinforced transverse partial-joint-penetration (PJP) groove welds	Y_i	=	Gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i , N
R_{pt}	=	Web plastification factor corresponding to the tension flange yielding limit state	Y_t	=	Hole reduction coefficient, N
R_u	=	Required strength (LRFD)			
R_{wl}	=	Total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table 510.2.5			

Z	=	Plastic section modulus about the axis of bending, mm^3	b_s	=	Stiffener width for one-sided stiffeners, mm
Z_b	=	Branch plastic section modulus about the correct axis of bending, mm^3	d	=	Nominal fastener diameter, mm
$Z_{x,y}$	=	Plastic section modulus about the principal axes, mm^3	d	=	Full nominal depth of the section, mm
a	=	Clear distance between transverse stiffeners, mm	d	=	Full nominal depth of tee, mm
a	=	Distance between connectors in a built-up member, mm	d	=	Depth of rectangular bar, mm
a	=	Shortest distance from edge of pin hole to edge of member measured parallel to the direction of force, mm	d	=	Diameter, mm
a	=	Half the length of the non-welded root face in the direction of the thickness of the tension-loaded plate, mm	d	=	Pin diameter, mm
a_w	=	Ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components	d	=	Roller diameter, mm
b	=	Outside width of leg in compression, mm	d_b	=	Beam depth, mm
b	=	Full width of longest angle leg, mm	d_b	=	Nominal diameter (body or shank diameter), mm
b	=	Width of unstiffened compression element; for flanges of I-shaped members and tees, the width b is half the full-flange width, b_f ; for legs of angles and flanges of channels and zees, the width b is the full nominal dimension; for plates, the width b is the distance from the free edge to the first row of fasteners or line of welds, or the distance between adjacent lines of fasteners or lines of welds; for rectangular HSS, the width b is the clear distance between the webs less the inside corner radius on each side, mm	d_c	=	Column depth, mm
b	=	Width of the angle leg resisting the shear force, mm	e	=	Eccentricity in a truss connection, positive being away from the branches, mm
b_{cf}	=	Width of column flange, mm	e_{mid-ht}	=	Distance from the edge of stud shank to the steel deck web, measured at mid-height of the deck rib, and in the load bearing direction of the stud (in other words, in the direction of maximum moment for a simply supported beam), mm
b_e	=	Reduced effective width, mm	f_a	=	Required axial stress at the point of consideration using LRFD or ASD load combinations, MPa
b_{eff}	=	Effective edge distance; the distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force, mm	$f_{b(w,z)}$	=	Required flexural stress at the point of consideration (major axis, minor axis) using LRFD or ASD load combinations, MPa
b_{eoi}	=	Effective width of the branch face welded to the chord	f'_c	=	Specified minimum compressive strength of concrete, MPa
b_{eov}	=	Effective width of the branch face welded to the overlapped brace	f'_{cm}	=	Specified minimum compressive strength of concrete at elevated temperatures, MPa
b_f	=	Flange width, mm	f_o	=	Stress due to D + R (the nominal dead load + the nominal load due to rainwater or snow exclusive of the ponding contribution), MPa
b_{fc}	=	Compression flange width, mm	f_v	=	Required shear strength per unit area, MPa
b_{ft}	=	Width of tension flange, mm	g	=	Transverse center-to-center spacing (gage) between fastener gage lines, mm
b_l	=	Longer leg of angle, mm	g	=	Gap between toes of branch members in a gapped K-connection, neglecting the welds, mm
b_s	=	Shorter leg of angle, mm	h	=	Clear distance between flanges less the fillet or corner radius for rolled shapes; for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, mm
			h	=	Distance between centroids of individual components perpendicular to the member axis of buckling, mm

h_c	=	Twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, mm			
h_o	=	Distance between flange centroids, mm		r_{ts}	= Effective radius of gyration used in the determination of L_r for the lateral-torsional buckling limit state for major axis bending of doubly symmetric compact I-shaped members and channels
h_p	=	Twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used, mm		r_x	= Radius of gyration about geometric axis parallel to connected leg, mm
h_{sc}	=	Hole factor		r_y	= Radius of gyration about y-axis, mm
j	=	Factor defined by Eq. 507.2-6 for minimum moment of inertia for a transverse stiffener		r_z	= Radius of gyration for the minor principal axis, mm
k	=	Distance from outer face of flange to the web toe of fillet, mm		s	= Longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm
k	=	Outside corner radius of the HSS, which is permitted to be taken as 1.5t if unknown, mm		t	= Thickness of element, mm
k_c	=	Coefficient for slender unstiffened elements, mm coefficient		t	= Wall thickness, mm
k_v	=	Web plate buckling coefficient		t	= Angle leg thickness, mm
l	=	Largest laterally unbraced length along either flange at the point of load, mm		t	= Width of rectangular bar parallel to axis of bending, mm
l	=	Length of bearing, mm		t	= Thickness of connected material, mm
l	=	Length of connection in the direction of loading, mm		t	= Thickness of plate, mm
n	=	Number of nodal braced points within the span		t	= Design wall thickness for HSS equal to 0.93 times the nominal wall thickness for ERW HSS and equal to the nominal wall thickness for SAW HSS, mm
n	=	Threads per mm		t	= Total thickness of fillers, mm
p	=	Ratio of element i deformation to its deformation at maximum stress		t	= Design wall thickness of HSS main member, mm
p	=	Projected length of the overlapping branch on the chord		t_b	= Design wall thickness of HSS branch member, mm
q	=	Overlap length measured along the connecting face of the chord beneath the two branches		t_{bi}	= Thickness of the overlapping branch, mm
r	=	Governing radius of gyration, mm		t_{bi}	= Thickness of the overlapped branch, mm
r_{cit}	=	Distance from instantaneous center of rotation to weld element with minimum $\Delta u/r_i$ ratio, mm		t_{cf}	= Thickness of the column flange, mm
r_i	=	Minimum radius of gyration of individual component in a built-up member, mm		t_f	= Thickness of the loaded flange, mm
r_{ib}	=	Radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, mm		t_f	= Flange thickness of channel shear connector, mm
$\bar{r}_o$	=	Polar radius of gyration about the shear center, mm		t_{fc}	= Compression flange thickness, mm
r_t	=	Radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone		t_p	= Thickness of plate, mm
				t_p	= Thickness of tension loaded plate, mm
				t_p	= Thickness of the attached transverse plate, mm
				t_s	= Web stiffener thickness, mm
				t_w	= Web thickness of channel shear connector, mm
				t_w	= Beam web thickness, mm
				t_w	= Web thickness, mm
				t_w	= Column web thickness, mm
				t_w	= Thickness of element, mm
				w	= Width of cover plate, mm
				w	= Weld leg size, mm
				w	= Subscript relating symbol to major principal axis bending
				w	= Plate width, mm

w	=	Leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, mm
w_c	=	Weight of concrete per unit volume
w_r	=	Average width of concrete rib or haunch, mm
x	=	Subscript relating symbol to strong axis
x_o, y_o	=	Coordinates of the shear center with respect to the centroid, mm
$\bar{x}$	=	Connection eccentricity, mm
y	=	Subscript relating symbol to weak axis
z	=	Subscript relating symbol to minor principal axis bending
α	=	Factor used in B_2 equation
α	=	Separation ratio for built-up compression members
β	=	Reduction factor given by Eq. 510.2-1
β	=	Width ratio; the ratio of branch diameter to chord diameter for round HSS; the ratio of overall branch width to chord width for rectangular HSS
β_T	=	Brace stiffness requirement excluding web distortion, N-mm/radian
β_{br}	=	Required brace stiffness
β_{eff}	=	Effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width
β_{eop}	=	Effective outside punching perimeter
β_{sec}	=	Web distortional stiffness, including the effect of web transverse stiffeners, if any, N-mm/radian
β_w	=	Section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression
Δ	=	First-order interstory drift due to the design loads, mm
Δ_H	=	First-order interstory drift due to lateral forces, mm
Δ_i	=	Deformation of welded elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i , mm
Δ_m	=	Deformation of weld element at maximum stress, mm
Δ_u	=	Deformation of welded element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, mm
γ	=	Chord slenderness ratio; the ratio of one-half the diameter to the wall thickness for round HSS; the ratio of one-half the width to wall thickness for rectangular HSS
ζ	=	Gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord for rectangular HSS
η	=	Load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width
λ	=	Slenderness parameter
λ_p	=	Limiting slenderness parameter for compact element
λ_{pf}	=	Limiting slenderness parameter for compact flange
λ_{pw}	=	Limiting slenderness parameter for compact web
λ_r	=	Limiting slenderness parameter for noncompact element
λ_{rf}	=	Limiting slenderness parameter for noncompact flange
λ_{rw}	=	Limiting slenderness parameter for noncompact web
μ	=	Mean slip coefficient for class A or B surfaces, as applicable, or as established by tests
ϕ	=	Resistance factor, specified in Section 502 through 511
ϕ_B	=	Resistance factor for bearing on concrete
ϕ_b	=	Resistance factor for flexure
ϕ_c	=	Resistance factor for compression
ϕ_c	=	Resistance factor for axially loaded composite columns
ϕ_{sf}	=	Resistance factor for shear on the failure path
ϕ_T	=	Resistance factor for torsion
ϕ_t	=	Resistance factor for tension
ϕ_v	=	Resistance factor for shear
Ω	=	Safety factor
Ω_B	=	Safety factor for bearing on concrete
Ω_b	=	Safety factor for flexure
Ω_c	=	Safety factor for compression
Ω_c	=	Safety factor for axially loaded composite columns
Ω_{sf}	=	Safety factor for shear on the failure path
Ω_t	=	Safety factor for torsion
Ω_t	=	Safety factor for tension
Ω_v	=	Safety factor for shear
ρ_{sr}	=	Minimum reinforcement ratio for longitudinal reinforcing element
θ	=	Angle of loading measured from the welded longitudinal axis, degrees
θ	=	Acute angle between the branch and chord, degrees
ϵ_{cu}	=	Strain corresponding to compressive strength, f'_c
τ_b	=	Parameter for reduced flexural stiffness using the direct analysis method

DEFINITIONS

ACTIVE FIRE PROTECTION is a building material and systems that are activated by a fire to mitigate adverse effects or to notify people to take some action mitigate adverse effects.

ADJUSTED BRACE STRENGTH is the strength of a brace in a buckling-restrained braced frame at deformations corresponding to 2.0 times the design story drift.

ALLOWABLE STRENGTH is the nominal strength divided by the safety factor, R_n/Ω .

ALLOWABLE STRESS is the allowable strength divided by the appropriate section property, such as section modulus or cross-section area.

AMPLIFICATION FACTOR is the multiplier of the results of first-order analysis to reflect second-order effects.

AMPLIFIED SEISMIC LOAD is the horizontal component of earthquake load E multiplied by Ω_o , where E and the horizontal component of E are specified in the NSCP code.

APPLICABLE BUILDING CODE is a Building Code under which the structure is designed (i.e NSCP 6th Edition).

ASD (ALLOWABLE STRENGTH DESIGN) is a method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations.

ASD LOAD COMBINATION refers to load combination in the NSCP code intended for allowable strength design (allowable stress design).

AUTHORITY HAVING JURISDICTION (AHJ) is the organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this standard.

AVAILABLE STRENGTH is the design strength or allowable strength, as appropriate.

AVAILABLE STRESS is the design stress or allowable stress, as appropriate.

AVERAGE RIB WIDTH is the average width of the rib of a corrugation in a formed steel deck.

BATTEN PLATE is a plate rigidly connected to two parallel components of a built-up column or beam designed to transmit shear between the components.

BEAM is a structural member that has the primary function of resisting bending moments.

BEAM-COLUMN is a structural member that resists both axial force and bending moment.

BEARING is the connection, limit state of ultimate shear forces transmitted by the mechanical fastener to the connection elements.

BEARING refers to a bolted connection, limit state of shear forces transmitted by the bolt to the connection elements.

BEARING (LOCAL COMPRESSIVE YIELDING) refers to the limit state of local compressive yielding due to the action of a member bearing against another member or surface.

BEARING-TYPE CONNECTION is a bolted connection where shear forces are transmitted by the bolt bearing against the connection elements.

BLOCK SHEAR RUPTURE refers to a connection, limit state of tension fracture along one path and shear yielding or shear fracture along another path.

BOUNDARY MEMBER is a portion along wall and diaphragm edge strengthened with structural steel sections and/or longitudinal steel reinforcement and transverse reinforcement.

BRACED FRAME is an essentially vertical truss system that provides resistance to lateral loads and provides stability for the structural system.

BRACING is a member or system that provides stiffness and strength to limit the out-of-plane movement of another member at a brace point.

BRACE TEST SPECIMEN is a single buckling-restrained brace element used for laboratory testing intended to model the brace in the Prototype.

BRANCH FACE is the wall of HSS branch member.

BRANCH MEMBER refers HSS connections, member that terminates at a chord member or main member.

BUCKLING is a limit state of sudden change in the geometry of a structure or any of its elements under a critical loading condition.

BUCKLING STRENGTH is a nominal strength for instability limits states.

BUCKLING-RESTRAINED BRACED FRAME (BRBF) is a diagonally braced frame satisfying the requirements of Section 529 in which all members of the bracing system are subjected primarily to axial forces and in which the limit state of compression buckling of braces is precluded at forces and deformations corresponding to 2.0 times the design story drift.

BUCKLING-RESTRAINING SYSTEM is the system of restraints that limits buckling of the steel core in BRBF. This system includes the casing on the steel core and structural elements adjoining its connections. The buckling-restraining system is intended to permit the transverse expansion and longitudinal contraction of the steel core for deformations corresponding to 2.0 times the design story drift.

BUILT-UP MEMBER, CROSS-SECTION, SECTION, SHAPE refers to the member, cross-section, section or shape fabricated from structural steel elements that are welded or bolted together.

CAMBER is a curvature fabricated into a beam or truss so as to compensate for deflection induced by loads.

CASING is an element that resists forces transverse to the axis of the brace thereby restraining buckling of the core. The casing requires a means of delivering this force to the remainder of the buckling-restraining system. The casing resists little or no force in the axis of the brace.

CHARPY V-NOTCH IMPACT TEST is a standard dynamic test measuring notch toughness of a specimen.

CHORD MEMBER refers to or HSS, primary member that extends through a truss connection.

CLADDING is the exterior covering of structure.

COLD-FORMED STEEL STRUCTURAL MEMBER refers to the shape manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold- or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

COLLECTOR ELEMENT refers to the member that serves to transfer loads between floor diaphragms and the members of the seismic load resisting system.

COLUMN is the structural member that has the primary function of resisting axial force.

COLUMN BASE is an assemblage of structural shapes, plates, connectors, bolts, and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.

COMBINED SYSTEM refers to the structure comprised of two or more lateral load-resisting systems of different type.

COMPACT SECTION is a section capable of developing a fully plastic stress distribution and possessing a rotation capacity of approximately three before the onset of local buckling.

COMPARTMENTATION is the enclosure of a building space with elements that have specific fire endurance.

COMPLETE-JOINT-PENETRATION GROOVE WELD (CJP) is a groove weld in which weld metal extends through the joint thickness, except as permitted for HSS connections.

COMPOSITE is the condition in which steel and concrete elements and members work as a unit in the distribution of internal forces.

COMPOSITE BEAM refers to structural steel beam in contact with and acting compositely with reinforced concrete via bond or shear connectors.

COMPOSITE BRACE is a reinforced-concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section used as a brace.

COMPOSITE COLUMN is a reinforced-concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section used as a column.

COMPOSITE ECCENTRICALLY BRACED FRAME (C-EBF) is a composite braced frame meeting the requirements of Section 545.

COMPOSITE INTERMEDIATE MOMENT FRAME (C-IMF) is a composite moment frame meeting the requirements of Section 541.

COMPOSITE ORDINARY BRACED FRAME (C-OBF) is a composite braced frame meeting the requirements of Section 544.

COMPOSITE ORDINARY MOMENT FRAME (C-OMF) is a composite moment frame meeting the requirements of Section 545.

COMPOSITE PARTIALLY RESTRAINED MOMENT FRAME (C-PRMF) is a composite moment frame meeting the requirements of Section 539.

COMPOSITE SHEAR WALL is a reinforced concrete wall that has unencased or reinforced-concrete encased structural steel sections as boundary members.

COMPOSITE SLAB) is a concrete slab supported on and bonded to a formed steel deck that acts as a diaphragm to transfer load to and between elements of the seismic load resisting system.

COMPOSITE SPECIAL CONCENTRICALLY BRACED FRAME (C-CBF) is a composite braced frame meeting the requirements of Section 543.

COMPOSITE SPECIAL MOMENT FRAME (C-SMF) is a composite moment frame meeting the requirements of Section 540.

COMPOSITE STEEL PLATE SHEAR WALL (C-SPW) is a wall consisting of steel plate with reinforced concrete encasement on one or both sides that provides out-of-plane stiffening to prevent buckling of the steel plate and meeting the requirements of Section 548.

CONCRETE CRUSHING is the limit state of compressive failure in concrete having reached the ultimate strain.

CONCRETE HAUNCH is a section of solid concrete that results from stopping the deck on each side of the girder in a composite floor system constructed using a formed steel deck.

CONCRETE-ENCASED BEAM is a beam that totally encased in concrete cast integrally with the slab.

CONFIRMATORY TEST is a test made, when desired, on members, connections, and assemblies designed in accordance with the provisions of Section 551 through Section 557, Appendices 1 and 2, and Section C-3 of this Specification or its specific references, in order to compare actual to calculated performance.

CONNECTION is a combination of structural elements and joints used to transmit forces between two or more members.

CONTINUITY PLATES is a column stiffeners at the top and bottom of the panel zone; also known as transverse stiffeners.

CONSTRUCTION DOCUMENTS refers to the design drawings, specifications, shop drawings, and erection drawings.

CONTRACTOR is a fabricator or erector, as applicable.

CONVECTIVE HEAT TRANSFER is the transfer of thermal energy from a point of higher temperature to a point of lower temperature through the motion of an intervening medium.

COPE is a cut-out made in a structural member to remove a flange and conform to the shape of an intersecting member.

COUPLING BEAM is a structural steel or composite beam connecting adjacent reinforced concrete wall elements so that they act together to resist lateral loads.

COVER PLATE is a plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus, or moment of inertia.

CROSS CONNECTION refers to HSS connection in which forces in branch members or connecting elements transverse to the main member are primarily equilibrated by forces in other branch members or connecting elements on the opposite side of the main member.

CROSS-SECTIONAL AREA is an Effective area, A_e , calculated using the effective widths of component elements in accordance with Section 552. If the effective widths of all component elements, determined in accordance with Section 552, are equal to the actual flat widths, it equals the gross or net area, as applicable.

CURTAIN WALL STUD is a member in the steel framed exterior wall system that transfers transverse (out-of-plane) loads and is limited to a superimposed axial load, exclusive of sheathing materials, of not more than 1460 N/m, or superimposed axial load of not more than 890 N per stud.

DEMAND CRITICAL WELD weld so designated by this chapter.

DESIGN-BASIS FIRE is a set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.

DESIGN EARTHQUAKE refers to the earthquake represented by the design response spectrum as specified in the NSCP code.

DESIGN LOAD is the applied load determined in accordance with either LRFD load combinations or ASD load combinations, whichever is applicable.

DESIGN METHODOLOGY is a set of step-by-step procedures, based on calculation or experiment, used to determine sizes, lengths, and details in the design of buckling-restrained braces and their connections.

DESIGN STORY DRIFT is an amplified story drift (drift under the design earthquake, including the effects of inelastic action), determined as specified in the NSCP code.

DESIGN STRENGTH is the resistance factor multiplied by the nominal strength, ϕR_n .

DESIGN STRESS RANGE refers to magnitude of change in stress due to the repeated application and removal of service live loads. For locations subject to stress reversal, it is the algebraic difference of the peak stresses.

DESIGN STRESS is the design strength divided by the appropriate section property, such as section modulus or cross section area.

DESIGN WALL THICKNESS is the HSS wall thickness assumed in the determination of section properties.

DIAGONAL BRACING is an inclined structural members carrying primarily axial load that are employed to enable a structural frame to act as a truss to resist lateral loads.

DIAGONAL STIFFENER is a web stiffener at column panel zone oriented diagonally to the flanges, on one or both sides of the web.

DIAPHRAGM is the roof, floor or other membrane or bracing system that transfers in-plane forces to the lateral force resisting system.

DIAPHRAGM PLATE is a plate possessing in-plane shear stiffness and strength, used to transfer forces to the supporting elements.

DIRECT ANALYSIS METHOD is the design method for stability that captures the effects of residual stresses and initial out-of-plumbness of frames by reducing stiffness and applying notional loads in a second-order analysis.

DIRECT BOND INTERACTION is a mechanism by which force is transferred between steel and concrete in a composite section by bond stress.

DIRECT STRENGTH METHOD is an alternative design method detailed in Section C-1 that provides predictions of member strengths without the use of effective widths.

DISTORTIONAL BUCKLING is a mode of buckling involving change in cross-sectional shape, excluding local buckling.

DISTORTIONAL FAILURE is a limit state of an HSS truss connection based on distortion of a rectangular HSS chord member into a rhomboidal shape.

DISTORTIONAL STIFFNESS is an out-of-plane flexural stiffness of web.

DOUBLE CURVATURE is a deformed shape of a beam with one or more inflection points within the span.

DOUBLE-CONCENTRATED FORCES is the two equal and opposite forces that form a couple on the same side of the loaded member.

DOUBLER is a plate added to, and parallel with, a beam or column web to increase resistance to concentrated forces.

DOUBLY-SYMMETRIC SECTION is a section symmetric about two orthogonal axes through its centroid.

DUAL SYSTEM is a structural system with the following features: (1) an essentially complete space frame that provides support for gravity loads; (2) resistance to lateral load provided by moment frames (SMF, IMF or OMF) that are capable of resisting at least 25 percent of the base shear, and concrete or steel shear walls, or steel braced frames (EBF, SCBF or OCBF); and (3) each system designed to resist the total lateral load in proportion to its relative rigidity.

DUCTILE LIMIT STATE includes member and connection yielding, bearing deformation at bolt holes, as well as buckling of members that conform to the width-thickness limitations of Table 521-1. Fracture of a member or of a connection, or buckling of a connection element, is not a ductile limit state.

ECCENTRICALLY BRACED FRAME (EBF) is a diagonally braced frame meeting the requirements of Section 528 that has at least one end of each bracing member connected to a beam a short distance from another beam-to-brace connection or a beam-to-column connection.

EFFECTIVE DESIGN WIDTH (EFFECTIVE WIDTH) is a flat width of an element reduced for design purposes, also known simply as the effective width.

EFFECTIVE LENGTH is the length of an otherwise identical column with the same strength when analyzed with pinned end conditions.

EFFECTIVE LENGTH FACTOR is the ratio between the effective length and the unbraced length of the member.

EFFECTIVE NET AREA is a net area modified to account for the effect of shear lag.

EFFECTIVE SECTION MODULUS is a section modulus reduced to account for buckling of slender compression elements.

EFFECTIVE WIDTH refers to reduced width of a plate or slab with an assumed uniform stress distribution which produces the same effect on the behavior of a structural member as the actual plate or slab width with its non-uniform stress distribution.

ELASTIC ANALYSIS is a structural analysis based on the assumption that the structure returns to its original geometry on removal of the load.

ELEVATED TEMPERATURES refers to heating conditions experienced by building elements or structures as a result of fire, which are in excess of the anticipated ambient conditions.

ENCASED COMPOSITE BEAM is a composite beam completely enclosed in reinforced concrete.

ENCASED COMPOSITE COLUMN is a structural steel column (rolled or built-up) completely encased in reinforced concrete.

ENCASED COMPOSITE COLUMN is a composite column consisting of a structural concrete column and one or more embedded steel shapes.

END PANEL is a web panel with an adjacent panel on one side only.

END RETURN is a length of fillet weld that continues around a corner in the same plane.

ENGINEER OF RECORD is a licensed professional responsible for sealing the design drawings and specifications.

EXEMPTED COLUMN refers to the column not meeting the requirements of Equation 522-3 for SMF.

EXPANSION ROCKER refers to support with curved surface on which a member bears that can tilt to accommodate expansion.

EXPANSION ROLLER is a round steel bar on which a member bears that can roll to accommodate expansion.

EXPECTED TENSILE STRENGTH is the tensile strength of a member, equal to the specified minimum tensile strength, F_u , multiplied by R_t .

EXPECTED YIELD STRENGTH is the yield strength in tension of a member, equal to the expected yield stress multiplied by A_g .

EXPECTED YIELD STRESS is the yield stress of the material, equal to the specified minimum yield stress, F_y , multiplied by R_y .

EYEBAR refers to pin-connected tension member of uniform thickness, with forged or thermally cut head of greater width than the body, proportioned to provide approximately equal strength in the head and body.

FACE BEARING PLATES refers to stiffeners attached to structural steel beams that are embedded in reinforced concrete walls or columns. The plates are located at the face of the reinforced concrete to provide confinement and to transfer loads to the concrete through direct bearing.

FACTORED LOAD is the product of a load factor and the nominal load.

FASTENER is a generic term for bolts, rivets, or other connecting devices.

FATIGUE is a limit state of crack initiation and growth resulting from repeated application of live loads.

FAYING SURFACE is a contact surface of connection elements transmitting a shear force.

FILLED COMPOSITE COLUMN is a round or rectangular structural steel section filled with concrete.

FILLED COMPOSITE COLUMN is a composite column consisting of a shell of HSS or steel pipe filled with structural concrete.

FILLER is a plate used to build up the thickness of one component.

FILLER METAL is a metal or alloy to be added in making a welded joint.

FILLET WELD is a weld of generally triangular cross section made between intersecting surfaces of elements.

FILLET WELD REINFORCEMENT is a fillet welds added to groove welds.

FIRE refers to as destructive burning, as manifested by any or all of the following: light, flame, heat, or smoke.

FIRE BARRIER is an element of construction formed of fire-resisting materials and tested in accordance with ASTM Standard E119, or other approved standard fire resistance test, to demonstrate compliance with the Building Code.

FIRE ENDURANCE is a measure of the elapsed time during which a material or assembly continues to exhibit fire resistance.

FIRE RESISTANCE is the property of assemblies that prevents or retards the passage of excessive heat, hot gases or flames under conditions of use and enables them to continue to perform a stipulated function.

FIRE RESISTANCE RATING is the period of time a building element, component or assembly maintains the ability to contain a fire, continues to perform a given structural function, or both, as determined by test or methods based on tests.

FIRST-ORDER ANALYSIS is a structural analysis in which equilibrium conditions are formulated on the undeformed structure; second-order effects are neglected.

FITTED BEARING STIFFENER is a stiffener used at a support or concentrated load that fits tightly against one or both flanges of a beam so as to transmit load through bearing.

FLANGE OF A SECTION IN BENDING (FLANGE) is a flat width of flange including any intermediate stiffeners plus adjoining corners.

FLARE BEVEL GROOVE WELD is a weld in a groove formed by a member with a curved surface in contact with a planar member.

FLARE V-GROOVE WELD is a weld in a groove formed by two members with curved surfaces.

FLASHOVER is the rapid transition to a state of total surface involvement in a fire of combustible materials within an enclosure.

FLAT WIDTH is a nominal width of rectangular HSS minus twice the outside corner radius. In absence of knowledge of the corner radius, the flat width may be taken as the total section width minus three times the thickness.

FLAT WIDTH is the Nominal width of an element exclusive of corners measured along its plane.

FLAT-WIDTH-TO THICKNESS RATIO (FLAT WIDTH RATIO) refers to flat width of an element measured along its plane, divided by its thickness.

FLEXURAL BUCKLING is a buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

FLEXURAL-TORSIONAL BUCKLING is a buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

FORCE is a resultant of distribution of stress over a prescribed area.

FORMED SECTION see cold-formed steel structural member.

FORMED STEEL DECK refers to composite construction, steel cold formed into a decking profile used as a permanent concrete form.

FULL, UNREDUCED AREA, A , is calculated without considering local buckling in the component elements, which equals either the gross area or net area, as applicable.

FULLY COMPOSITE BEAM is a composite beam that has a sufficient number of shear connectors to develop the nominal plastic flexural strength of the composite section.

FULLY RESTRAINED MOMENT CONNECTION is a connection capable of transferring moment with negligible rotation between connected members.

GAGE is a transverse center-to-center spacing of fasteners.

GAP CONNECTION refers to HSS truss connection with a gap or space on the chord face between intersecting branch members.

GENERAL COLLAPSE. is a limit state of chord plastification of opposing sides of a round HSS chord member at a cross-connection.

GEOMETRIC AXIS is the axis parallel to web, flange or angle leg.

GIRDER see Beam.

GIRDER see Beam.

GIRDER FILLER is a narrow piece of sheet steel used as a fill between the edge of a deck sheet and the flange of a girder in a composite floor system constructed using a formed steel deck.

GIRT is a horizontal structural member that supports wall panels and is primarily subjected to bending under horizontal loads, such as wind load.

GOUGE refers to relatively smooth surface groove or cavity resulting from plastic deformation or removal of material.

GRAVITY AXIS is the axis through the center of gravity of a member along its length.

GRAVITY FRAME is a portion of the framing system not included in the lateral load resisting system.

GRAVITY LOAD is a load, such as that produced by dead and live loads, acting in the downward direction.

GRIP (OF BOLT) is the thickness of material through which a bolt passes.

GROSS AREA, A_g , without deductions for holes, openings and cutouts.

GROOVE WELD is a weld in a groove between connection elements. See also AWS D1.1.

GUSSET PLATE is a plate element connecting truss members or a strut or brace to a beam or column.

HEAT FLUX is the radiant energy per unit surface area.

HEAT RELEASE RATE is the rate at which thermal energy is generated by a burning material.

HORIZONTAL SHEAR is a Force at the interface between steel and concrete surfaces in a composite beam.

HSS is a square, rectangular or round hollow structural steel section produced in accordance with a pipe or tubing product specification.

User Note: A pipe can be designed using the same design rules for round HSS sections as long as it conforms to ASTM A53 Class B and the appropriate parameters are used in the design.

INELASTIC ANALYSIS refers to structural analysis that takes into account inelastic material behavior, including plastic analysis.

INELASTIC DEFORMATION is the permanent or plastic portion of the axial displacement in a buckling-restrained brace.

IN-PLANE INSTABILITY. refers to limit state of a beam-column bent about its major axis while lateral buckling or lateral-torsional buckling is prevented by lateral bracing.

IN-PLANE INSTABILITY refers to buckling involving in the plane of the frame or the member.

INTERMEDIATE MOMENT FRAME (IMF) is the moment frame system that meets the requirements of Section 523.

INTERMEDIATE SEISMIC SYSTEMS is seismic systems designed assuming moderate inelastic action occurs in some members under the design earthquake.

INTERSTORY DRIFT ANGLE. refers to interstory displacement divided by story height, radians.

INSTABILITY refers to limit state reached in the loading of a structural component, frame or structure in which a slight disturbance in the loads or geometry produces large displacements.

INVERTED-V-BRACED FRAME. refers to V-braced frame.

JOINT is an area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and the method of force transfer.

JOINT ECCENTRICITY refers to HSS truss connection, perpendicular distance from chord member center of gravity to intersection of branch member work points.

K-AREA is the region of the web that extends from the tangent point of the web and the flange-web fillet (AISC “k” dimension) a distance of 38 mm into the web beyond the “k” dimension.

K-BRACED FRAME is a bracing configuration in which braces connect to a column at a location with no diaphragm or other out-of-plane support.

K-CONNECTION refers to HSS connection in which forces in branch members or connecting elements transverse to the main member are primarily equilibrated by forces in other branch members or connecting elements on the same side of the main member.

LACING is a plate, angle or other steel shape, in a lattice configuration, that connects two steel shapes together.

LAP JOINT is a joint between two overlapping connection elements in parallel planes.

LATERAL BRACING is a diagonal bracing, shear walls or equivalent means for providing in-plane lateral stability.

LATERAL BRACING MEMBER refers to a member that is designed to inhibit lateral buckling or lateral-torsional buckling of primary framing members.

LATERAL LOAD refers to load, such as that produced by wind or earthquake effects, acting in a lateral direction.

LATERAL LOAD RESISTING SYSTEM refers to structural system designed to resist lateral loads and provide stability for the structure as a whole.

LATERAL-TORSIONAL BUCKLING is a buckling mode of a flexural member involving deflection normal to the plane of bending occurring simultaneously with twist about the shear center of the cross-section.

LEANING COLUMN is a column designed to carry gravity loads only, with connections that are not intended to provide resistance to lateral loads.

LENGTH EFFECTS is the consideration of the reduction in strength of a member based on its unbraced length.

LIMIT STATE is the condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to have reached its ultimate load-carrying capacity (strength limit state).

LINK in EBF, the segment of a beam that is located between the ends of two diagonal braces or between the end of a diagonal brace and a column. The length of the link is defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face.

LINK INTERMEDIATE WEB STIFFENERS is the vertical web stiffeners placed within the link in EBF.

LINK ROTATION ANGLE is an inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift.

LINK SHEAR DESIGN STRENGTH refers to lesser of the available shear strength of the link developed from the moment or shear strength of the link.

LOAD refers to force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

LOAD-CARRYING REINFORCEMENT is a reinforcement in composite members designed and detailed to resist the required loads.

LOAD is a force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

LOAD EFFECT refers to forces, stresses and deformations produced in a structural component by the applied loads.

LOAD FACTOR is a factor that accounts for deviations of the nominal load from the actual load, for uncertainties in the analysis that transforms the load into a load effect and for the probability that more than one extreme load will occur simultaneously.

LOCAL BENDING is the ultimate state of large deformation of a flange under a concentrated transverse force.

LOCAL BUCKLING refers to buckling of a compression element where the line junctions between elements remain straight and angles between elements do not change.

LOCAL CRIPPLING refers to limit state of local failure of web plate in the immediate vicinity of a concentrated load or reaction.

LOCAL YIELDING refers to yielding that occurs in a local area of an element.

LOWEST ANTICIPATED SERVICE TEMPERATURE (LAST) is the lowest 1-hour average temperature with a 100-year mean recurrence interval.

LRFD (LOAD AND RESISTANCE FACTOR DESIGN) is the method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations.

LRFD LOAD COMBINATION is the load combination in the NSCP code intended for strength design (load and resistance factor design).

MAIN MEMBER refers to HSS connections, chord member, column or other HSS member to which branch members or other connecting elements are attached.

MASTER COIL refers to one continuous, weld-free coil as produced by a hot mill, cold mill, metallic coating line or paint line and identifiable by a unique coil number. In some cases, this coil is cut into smaller coils or slit into narrower coils; however, all of these smaller and /or narrower finished coils are said to have come from the same master coil if they are traceable to the original master coil number.

MEASURED FLEXURAL RESISTANCE is a bending moment measured in a beam at the face of the column, for a beam-to-column test specimen tested in accordance with Appendix B-4.

MECHANISM is a structural system that includes a sufficient number of real hinges, plastic hinges or both, so as to be able to articulate in one or more rigid body modes.

MILL SCALE is an oxide surface coating on steel formed by the hot rolling process.

MILLED SURFACE is a surface that has been machined flat by a mechanically guided tool to a flat, smooth condition.

MOMENT CONNECTION is a connection that transmits bending moment between connected members.

MOMENT FRAME refers to framing system that provides resistance to lateral loads and provides stability to the structural system, primarily by shear and flexure of the framing members and their connections.

MULTIPLE-STIFFENED ELEMENT is an element stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners parallel to the direction of stress.

NET AREA, A_n , is equal to gross area less the area of holes, openings, and cutouts.

NODAL BRACE is a brace that prevents lateral movement or twist independently of other braces at adjacent brace points (see relative brace).

NOMINAL DIMENSION refers to the designated or theoretical dimension, as in the tables of section properties.

NOMINAL LOAD is the magnitude of the load specified by the NSCP code.

NOMINAL RIB HEIGHT refers to height of formed steel deck measured from the underside of the lowest point to the top of the highest point.

NOMINAL STRENGTH is a strength of a structure or component (without the resistance factor or safety factor applied) to resist load effects, as determined in accordance with this Specification.

NONCOMPACT SECTION is a section that can develop the yield stress in its compression elements before local buckling occurs, but cannot develop a rotation capacity of three.

NONDESTRUCTIVE TESTING refers to inspection procedure wherein no material is destroyed and integrity of the material or component is not affected.

NOTCH TOUGHNESS refers to energy absorbed at a specified temperature as measured in the Charpy V-Notch test.

NOTIONAL LOAD refers to Virtual load applied in a structural analysis to account for destabilizing effects that are not otherwise accounted for in the design provisions.

NSCP CODE refers to the building code under which the structure is designed. NSCP 6th Edition.

ORDINARY CONCENTRICALLY BRACED FRAME (OCBF) refers to a diagonally braced frame meeting the requirements of Section 527 in which all members of the bracing system are subjected primarily to axial forces.

ORDINARY MOMENT FRAME (OMF) refers to moment frame system that meets the requirements of Section 524.

ORDINARY REINFORCED CONCRETE SHEAR WALL WITH STRUCTURAL STEEL ELEMENTS (C-ORCW) is a composite shear walls meeting the requirements of Section 546.

ORDINARY SEISMIC SYSTEMS is a seismic systems designed assuming limited inelastic action occurs in some members under the design earthquake.

OUT-OF-PLANE BUCKLING refers to limit state of a beam-column bent about its major axis while lateral buckling or lateral-torsional buckling is not prevented by lateral bracing.

OVERLAP CONNECTION refers HSS truss connection in which intersecting branch members overlap.

OVERSTRENGTH FACTOR, Ω_o refers to the factor specified by the NSCP code in order to determine the amplified seismic load, where required by these Provisions.

PANEL ZONE is a web area of beam-to-column connection delineated by the extension of beam and column flanges through the connection, transmitting moment through a shear panel.

PARTIALLY COMPOSITE BEAM is an unencased composite beam with a nominal flexural strength controlled by the strength of the shear stud connectors.

PARTIALLY RESTRAINED COMPOSITE CONNECTION is a partially restrained (PR) connections as defined in the Specification that connect partially or fully composite beams to steel columns with flexural resistance provided by a force couple achieved with steel reinforcement in the slab and a steel seat angle or similar connection at the bottom flange.

PARTIAL-JOINT-PENETRATION GROOVE WELD (PJP) is a Groove weld in which the penetration is intentionally less than the complete thickness of the connected element.

PARTIALLY RESTRAINED MOMENT CONNECTION is a connection capable of transferring moment with rotation between connected members that is not negligible.

PASSIVE FIRE PROTECTION refers to Building materials and systems whose ability to resist the effects of fire does not rely on any outside activating condition or mechanism.

PERCENT ELONGATION is a measure of ductility, determined in a tensile test as the maximum elongation of the gage length divided by the original gage length.

PERFORMANCE-BASED DESIGN is an engineering approach to structural design that is based on agreed-upon performance goals and objectives, engineering analysis and quantitative assessment of alternatives against those design goals and objectives using accepted engineering tools, methodologies and performance criteria.

PERFORMANCE TEST is a test made on structural members, connections, and assemblies whose performance cannot be determined in accordance with Section 551 to Section 557 of this specification or its specific references.

PERMANENT LOAD refers to load in which variations over time are rare or of small magnitude. All other loads are variable loads.

PIPE see HSS.

PITCH is the longitudinal center-to-center spacing of fasteners. Center-to-center spacing of bolt threads along axis of bolt.

PLASTIC ANALYSIS refers to structural analysis based on the assumption of rigid-plastic behavior, in other words, that equilibrium is satisfied throughout the structure and the stress is at or below the yield stress.

PLASTIC HINGE refers to yielded zone that forms in a structural member when the plastic moment is attained. The member is assumed to rotate further as if hinged, except that such rotation is restrained by the plastic moment.

PLASTIC MOMENT refers to the theoretical resisting moment developed within a fully yielded cross section.

PLASTIC STRESS DISTRIBUTION METHOD is a method for determining the stresses in a composite member assuming that the steel section and the concrete in the cross section are fully plastic.

PLASTIFICATION refers to HSS connection, limit state based on an out-of-plane flexural yield line mechanism in the chord at a branch member connection.

PLATE GIRDER is a built-up beam.

PLUG WELD is a weld made in a circular hole in one element of a joint fusing that element to another element.

POINT-SYMMETRIC SECTION is a section symmetrical about a point (centroid) such as a Z-section having equal flanges.

PONDING is retention of water due solely to the deflection of flat roof framing.

POSITIVE FLEXURAL STRENGTH is the load or force that can be carried by an element, member, or frame after initial buckling has occurred.

POST-BUCKLING STRENGTH is the load or force that can be carried by an element, member, or frame after initial buckling has occurred.

PREQUALIFIED CONNECTION refers to the connection that complies with the requirements of Appendix B-1.

PRESCRIPTIVE DESIGN is a design method that documents compliance with general criteria established in a building code.

PRETENSIONED JOINT is a joint with high-strength bolts tightened to the specified minimum pretension.

PROPERLY DEVELOPED is a reinforcing bars detailed to yield in a ductile manner before crushing of the concrete occurs. Bars meeting the provisions of ACI 318 in so far as development length, spacing and cover shall be deemed to be properly developed.

PROTECTED ZONE is an area of members in which limitations apply to fabrication and attachments. See Section 520.4.

PROTOTYPE refers to the brace, connections, members, steel properties, and other design, detailing, and construction features to be used in the actual building frame.

PROTOTYPE is the connection or brace design that is to be used in the building (SMF, IMF, EBF, and BRBF).

PROVISIONS refers to this document, and in reference to the AISC Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341).

PRYING ACTION is an amplification of the tension force in a bolt caused by leverage between the point of applied load, the bolt and the reaction of the connected elements.

PUBLISHED SPECIFICATION are requirements for a steel listed by a manufacturer, processor, producer, purchaser, or other body, which (1) are generally available in the public domain or are available to the public upon request, (2) are established before the steel is ordered, and (3) as a minimum, specify minimum mechanical properties, chemical composition limits, and, if coated sheet, coating properties.

PUNCHING LOAD is a component of branch member force perpendicular to a chord.

PURLIN is a horizontal structural member that supports roof deck and is primarily subjected to bending under vertical loads such as snow, wind or dead loads.

$P - \delta$ EFFECT is the effect of loads acting on the deflected shape of a member between joints or nodes.

$P - \Delta$ EFFECT is the effect of loads acting on the displaced location of joints or nodes in a structure. In tiered building structures, this is the effect of loads acting on the laterally displaced location of floors and roofs.

QUALITY ASSURANCE is system of shop and field activities and controls implemented by the owner or his/her designated representative to provide confidence to the owner and the building authority that quality requirements are implemented.

QUALITY ASSURANCE PLAN is a written description of qualifications, procedures, quality inspections, resources, and records to be used to provide assurance that the structure complies with the engineer's quality requirements, specifications and contract documents.

QUALITY CONTROL is a system of shop and field controls implemented by the fabricator and erector to ensure that contract and company fabrication and erection requirements are met.

RATIONAL ENGINEERING ANALYSIS is an analysis based on theory that is appropriate for the situation, relevant test data if available, and sound engineering judgment.

REDUCED BEAM SECTION refers to reduction in cross section over a discrete length that promotes a zone of inelasticity in the member.

REENTRANT refers to a cope or weld access hole, a cut at an abrupt change in direction in which the exposed surface is concave.

REINFORCED-CONCRETE-ENCASED SHAPES is a structural steel sections encased in reinforced concrete.

RELATIVE BRACE is a brace that controls the relative movement of two adjacent brace points along the length of a beam or column or the relative lateral displacement of two stories in a frame (see nodal brace).

REQUIRED STRENGTH is the forces, stresses and deformations acting on the structural component, determined by either structural analysis, for the LRFD or ASD load combinations, as appropriate, or as specified by this Specification or Standard.

RESISTANCE. See the definition of Nominal Strength.

RESISTANCE FACTOR, ϕ is a factor that accounts for unavoidable deviations of the nominal strength from the actual strength and for the manner and consequences of failure.

RESTRAINED CONSTRUCTION refers to floor and roof assemblies and individual beams in buildings where the surrounding or supporting structure is capable of resisting substantial thermal expansion throughout the range of anticipated elevated temperatures.

RESTRAINING BARS is a steel reinforcement in composite members that is not designed to carry required loads, but is provided to facilitate the erection of other steel reinforcement and to provide anchorage for stirrups or ties. Generally, such reinforcement is not spliced to be continuous.

REVERSE CURVATURE see double curvature

ROOT OF JOINT is a portion of a joint to be welded where the members are closest to each other.

ROTATION CAPACITY is the incremental angular rotation that a given shape can accept prior to excessive load shedding, defined as the ratio of the inelastic rotation attained to the idealized elastic rotation at first yield.

RUPTURE STRENGTH is a strength limited by breaking or tearing of members or connecting elements.

RUPTURE STRENGTH refers to a connection, strength limited by tension or shear rupture.

SAFETY FACTOR, Ω is the factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the nominal load, uncertainties in the analysis that transforms the load into a load effect and for the manner and consequences of failure.

SECOND-ORDER ANALYSIS refers to the structural analysis in which equilibrium conditions are formulated on the deformed structure; second-order effects (both $P - \delta$ and $P - \Delta$, unless specified otherwise) are included.

SECOND-ORDER EFFECT is an effect of loads acting on the deformed configuration of a structure; includes $P - \delta$ effect and $P - \Delta$ effect.

SEISMIC DESIGN CATEGORY refers to the classification assigned to a building by the NSCP code based upon its seismic use group and the design spectral response acceleration coefficients.

SEISMIC LOAD RESISTING SYSTEM (SLRS) is an assembly of structural elements in the building that resists seismic loads, including struts, collectors, chords, diaphragms and trusses. Seismic response modification coefficient, R . Factor that reduces seismic load effects to strength level as specified by the NSCP code.

SEISMIC RESPONSE MODIFICATION COEFFICIENT is a factor that reduces seismic load effects to strength level.

SEISMIC USE GROUP refers to the classification assigned to a structure based on its use as specified by the NSCP code.

SERVICE LOAD is a load under which serviceability limit states are evaluated.

SERVICE LOAD COMBINATION is a load combination under which serviceability limit states are evaluated.

SERVICEABILITY LIMIT STATE is a limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability or the comfort of its occupants or function of machinery, under normal usage.

SHEAR BUCKLING is a buckling mode in which a plate element, such as the web of a beam, deforms under pure shear applied in the plane of the plate.

SHEAR CONNECTOR refers to headed stud, channel, plate or other shape welded to a steel member and embedded in concrete of a composite member to transmit shear forces at the interface between the two materials.

SHEAR CONNECTOR STRENGTH refers to limit state of reaching the strength of a shear connector, as governed by the connector bearing against the concrete in the slab or by the tensile strength of the connector.

SHEAR RUPTURE refers to limit state of rupture (fracture) due to shear.

SHEAR WALL is a wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system.

SHEAR YIELDING is a yielding that occurs due to shear.

SHEAR YIELDING (PUNCHING) refers to HSS connection, limit state based on out-of-plane shear strength of the chord wall to which branch members are attached.

SHEET STEEL refers to a composite floor system, steel used for closure plates or miscellaneous trimming in a formed steel deck.

SHIM is a thin layer of material used to fill a space between faying or bearing surfaces.

SIDESWAY BUCKLING is a limit state of lateral buckling of the tension flange opposite the location of a concentrated compression force.

SIDEWALL CRIPPLING (FRAME) is a limit state of web crippling of the sidewalls of a chord member at a HSS truss connection.

SIDEWALL CRUSHING is a limit state based on bearing strength of chord member sidewall in HSS truss connection.

SIMPLE CONNECTION is a connection that transmits negligible bending moment between connected members.

SINGLE-CONCENTRATED FORCE is a tensile or compressive force applied normal to the flange of a member.

SINGLE CURVATURE is a deformed shape of a beam with no inflection point within the span.

SINGLY-SYMMETRIC SECTION is a section symmetric about only one axis through its centroid.

SLENDER-ELEMENT SECTION is a cross section possessing plate components of sufficient slenderness such that local buckling in the elastic range will occur.

SLIP refers to in a bolted connection, limit state of relative motion of connected parts prior to the attainment of the available strength of the connection.

SLIP-CRITICAL CONNECTION refers to bolted connection designed to resist movement by friction on the faying surface of the connection under the clamping forces of the bolts.

SLOT WELD refers to weld made in an elongated hole fusing an element to another element.

SNUG-TIGHTENED JOINT is a joint with the connected plies in firm contact as specified in Section 510.

SPECIAL CONCENTRICALLY BRACED FRAME (SCBF) is a diagonally braced frame meeting the requirements of Section 526 in which all members of the bracing system are subjected primarily to axial forces.

SPECIAL MOMENT FRAME (SMF) refers to moment frame system that meets the requirements of Section 522.

SPECIAL PLATE SHEAR WALL (SPSW) refers to plate shear wall system that meets the requirements of Section 530.

SPECIAL REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS (C-SRCW) is a composite shear walls meeting the requirements of Section 547.

SPECIAL SEISMIC SYSTEMS is seismic systems designed assuming significant inelastic action occurs in some members under the design earthquake.

SPECIAL TRUSS MOMENT FRAME (STMF), refers to truss moment frame system that meets the requirements of Section 525.

SPECIFICATION refers to the AISC Specification for Structural Steel Buildings (ANSI/AISC 360).

SPECIFIED MINIMUM TENSILE STRENGTH is the lower limit of tensile strength specified for a material as defined by ASTM.

SPECIFIED MINIMUM YIELD STRESS is the lower limit of yield stress specified for a material as defined by ASTM.

SPLICE is a connection between two structural elements joined at their ends to form a single, longer element.

SS (STRUCTURAL STEEL) is an ASTM designation for certain steels intended for structural applications.

STABILITY is a condition reached in the loading of a structural component, frame or structure in which a slight disturbance in the loads or geometry does not produce large displacements.

STATIC YIELD STRENGTH is the strength of a structural member or connection determined on the basis of testing conducted under slow monotonic loading until failure.

STEEL CORE is an axial-force-resisting element of braces in BRBF. The steel core contains a yielding segment and connections to transfer its axial force to adjoining elements; it may also contain projections beyond the casing and transition segments between the projections and yielding segment.

STIFFENED ELEMENT refers to flat compression element with adjoining out-of-plane elements along both edges parallel to the direction of loading.

STIFFENED OR PARTIALLY STIFFENED COMPRESSION ELEMENTS. is a flat compression elements (i.e., a plane compression flange of a flexural member or a plane web or flange of compression member) of which both edges parallel to the direction of stresses are stiffened either by a web, flange, stiffening lip, intermediate stiffener, or the like.

STIFFENER is a structural element, usually an angle or plate, attached to a member to distribute load, transfer shear or prevent buckling.

STIFFNESS is a resistance to deformation of a member or structure, measured by the ratio of the applied force (or moment) to the corresponding displacement (or rotation).

STRAIN COMPATIBILITY METHOD is a method for determining the stresses in a composite member considering the stress-strain relationships of each material and its location with respect to the neutral axis of the cross section.

STRENGTH LIMIT STATE refers to limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached.

STRESS refers to stress as used in this Specification means force per unit area.

STRESS is a force per unit area caused by axial force, moment, shear or torsion.

STRESS CONCENTRATION is a localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) due to abrupt changes in geometry or localized loading.

STRONG AXIS is a major principal centroidal axis of a cross section.

STRUCTURAL ANALYSIS is a determination of load effects on members and connections based on principles of structural mechanics.

STRUCTURAL COMPONENT refers to member, connector, connecting element or assemblage.

STRUCTURAL MEMBERS See the definition of Cold-Formed Structural Steel Structural Members

STRUCTURAL STEEL are steel elements as defined in Section 2.1 of the AISC Code of Standard Practice for Steel Buildings and Bridges.

STRUCTURAL SYSTEM is an assemblage of load-carrying components that are joined together to provide interaction or interdependence.

SUB-ASSEMBLAGE TEST SPECIMEN is the combination of the brace, the connections and testing apparatus that replicate as closely as practical the axial and flexural deformations of the brace in the prototype.

SUB-ELEMENT OF A MULTIPLE STIFFENED ELEMENT is a portion of a multiple stiffened element between adjacent intermediate stiffeners, between web and intermediate stiffener, or between edge and intermediate stiffener.

T-CONNECTION refers to HSS connection in which the branch member or connecting element is perpendicular to the main member and in which forces transverse to the main member are primarily equilibrated by shear in the main member.

TENSILE RUPTURE refers to limit state of rupture (fracture) due to tension.

TENSILE STRENGTH (OF MATERIAL) is the maximum tensile stress that a material is capable of sustaining as defined by ASTM.

TENSILE STRENGTH (OF MEMBER) is the maximum tension force that a member is capable of sustaining.

TENSILE YIELDING is the yielding that occurs due to tension.

TENSION AND SHEAR RUPTURE refers to bolt, limit state of rupture (fracture) due to simultaneous tension and shear force.

TENSION FIELD ACTION is the behavior of a panel under shear in which diagonal tensile forces develop in the web and compressive forces develop in the transverse stiffeners in a manner similar to a Pratt truss.

TESTED CONNECTION refers to connection that complies with the requirements of Appendix B-4.

TEST SPECIMEN is a brace test specimen or sub-assembly test specimen.

THERMALLY CUT is a cut with gas, plasma or laser.

THICKNESS refers to thickness of any element or section is the base steel thickness, exclusive of coatings.

TIE PLATE is a plate element used to join two parallel components of a built-up column, girder or strut rigidly connected to the parallel components and designed to transmit shear between them.

TOE OF FILLET refers to junction of a fillet weld face and base metal. Tangent point of a rolled section fillet.

TORSIONAL BRACING is a bracing resisting twist of a beam or column.

TORSIONAL BUCKLING is a buckling mode in which a compression member twists about its shear center axis.

TORSIONAL YIELDING is the yielding that occurs due to torsion.

TRANSVERSE REINFORCEMENT refers to a steel reinforcement in the form of closed ties or welded wire fabric providing confinement for the concrete surrounding the steel shape core in an encased concrete composite column.

TRANSVERSE STIFFENER is a web stiffener oriented perpendicular to the flanges, attached to the web.

TUBING see HSS.

TURN-OF-NUT METHOD refers to procedure whereby the specified pretension in high-strength bolts is controlled by rotating the fastener component a predetermined amount after the bolt has been snug tightened.

UNBRACED LENGTH is a distance between braced points of a member, measured between the centers of gravity of the bracing members.

UNENCASED COMPOSITE BEAM is a composite beam wherein the steel section is not completely enclosed in reinforced concrete and relies on mechanical connectors for composite action with a reinforced slab or slab on metal deck.

UNEVEN LOAD DISTRIBUTION refers to HSS connection, condition in which the load is not distributed through the cross section of connected elements in a manner that can be readily determined.

UNFRAMED END is the end of a member not restrained against rotation by stiffeners or connection elements.

UNRESTRAINED CONSTRUCTION refers to the floor and roof assemblies and individual beams in buildings that are assumed to be free to rotate and expand throughout the range of anticipated elevated temperatures.

UNSTIFFENED COMPRESSION ELEMENTS is a flat compression element stiffened at only one edge parallel to the direction of stress.

UNSTIFFENED ELEMENT refers to flat compression element with an adjoining out-of-plane element along one edge parallel to the direction of loading.

UNSYMMETRIC SECTION is a section not symmetric either about an axis or a point.

V-BRACED FRAME is a concentrically braced frame (SCBF, OCBF or BRBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system is also referred to as an inverted-V-braced frame.

VARIABLE LOAD is a load not classified as permanent load.

VERTICAL BRACING SYSTEM is a system of shear walls, braced frames or both, extending through one or more floors of a building.

VIRGIN STEEL is steel as received from the steel producer or warehouse before being cold worked as a result of fabricating operations.

VIRGIN STEEL PROPERTIES is a mechanical property of virgin steel such as yield stress, tensile strength, and elongation.

WEAK AXIS is a minor principal centroidal axis of a cross section.

WEATHERING STEEL refers to high-strength, low-alloy steel that, with suitable precautions, can be used in normal atmospheric exposures (not marine) without protective paint coating.

WEB is the portion of the section that is joined to two flanges, or that is joined to only one flange provided it crosses the neutral axis.

WEB BUCKLING. refers to limit state of lateral instability of a web.

WEB COMPRESSION BUCKLING refers to limit state of out-of-plane compression buckling of the web due to a concentrated compression force.

WEB CRIPPLING is a limit state of local failure of web plate in the immediate vicinity of a concentrated load or reaction.

WEB SIDESWAY BUCKLING refers to limit state of lateral buckling of the tension flange opposite the location of a concentrated compression force.

WELD METAL is a portion of a fusion weld that has been completely melted during welding. Weld metal has elements of filler metal and base metal melted in the weld thermal cycle.

WELD ROOT See root of joint.

X-BRACED FRAME is a concentrically braced frame (OCBF or SCBF) in which a pair of diagonal braces crosses near the mid-length of the braces.

Y-BRACED FRAME is an eccentrically braced frame (EBF) in which the stem of the Y is the link of the EBF system.

Y-CONNECTION refers to HSS connection in which the branch member or connecting element is not perpendicular to the main member and in which forces transverse to the main member are primarily equilibrated by shear in the main member.

YIELD MOMENT refers to in a member subjected to bending, the moment at which the extreme outer fiber first attains the yield stress.

YIELD POINT refers to first stress in a material at which an increase in strain occurs without an increase in stress as defined by ASTM.

YIELD STRENGTH refers to stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.

YIELD STRESS is a generic term to denote either yield point or yield strength, as appropriate for the material.

YIELDING is a limit state of inelastic deformation that occurs after the yield stress is reached.

YIELDING (PLASTIC MOMENT) is a yielding throughout the cross section of a member as the bending moment reaches the plastic moment.

YIELDING (YIELD MOMENT) is a yielding at the extreme fiber on the cross section of a member when the bending moment reaches the yield moment

SECTION 501 GENERAL PROVISIONS

501.1 Scope

Section 501 states the scope of the specification, summarizes referenced specification, code, and standard documents, and provides requirements for materials and contract documents.

User Note: User notes are intended to provide concise and practical guidance in the application of the provisions.

This Chapter sets forth criteria for the design, fabrication, and erection of structural steel buildings and other structures, where other structures are defined as those structures designed, fabricated, and erected in a manner similar to buildings, with building-like vertical and lateral load resisting elements. Where conditions are not covered by this Chapter, designs are permitted to be based on tests or analysis, subject to the approval of the authority having jurisdiction. Alternate methods of analysis and design shall be permitted, provided such alternate methods or criteria are acceptable to the authority having jurisdiction.

User Note: For the design of structural members, other than Hollow Structural Sections (HSS), that are cold-formed to shapes, with elements not more than 25 mm in thickness, the provisions in the AISI North American Specification for the Design of Cold-Formed Steel Structural Members are recommended.

501.1.1 Low-Seismic Applications

When the seismic response modification coefficient, R , (as specified in this Chapter) is taken equal to or less than 3, the design, fabrication, and erection of structural-steel-framed buildings and other structures shall comply with this Chapter.

501.1.2 High-Seismic Applications

When the seismic response modification coefficient, R , (as specified in this Chapter) is taken greater than 3, the design, fabrication and erection of structural-steel-framed buildings and other structures shall comply with the requirements in the Seismic Provisions for Structural Steel Buildings (NSCP Chapter 5 Part 2), in addition to the provisions of this Specification.

501.1.3 Nuclear Applications

The design of nuclear structures shall comply with the requirements of the Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures in Nuclear Facilities (ANSI/AISC N690) including Supplement No. 2 or the Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities (ANSI/AISC N690L), in addition to the provisions of this Chapter.

501.2 Referenced Specifications, Codes and Standards

The following specifications, codes and standards are referenced in this Chapter:

American Concrete Institute (ACI)

ACI 318-08 *Building Code Requirements for Structural Concrete and Commentary*

ACI 318M-08 *Metric Building Code Requirements for Structural Concrete and Commentary*

ACI 349-06 *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*

American Institute of Steel Construction, Inc. (AISC)

AISC 3 03-10 *Code of Standard Practice for Steel Buildings and Bridges*

ANSI/AISC 341-10 *Seismic Provisions for Structural Steel Buildings*

ANSI/AISC N690-1994(R2004) *Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities*, including Supplement No. 2

ANSI/AISC N690L-03 *Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities*

American Society of Civil Engineers (ASCE)

SEI/ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*

ASCE/SFPE 29-05 *Standard Calculation Methods for Structural Fire Protection*

American Society of Mechanical Engineers (ASME)

ASME B 18.2.6-06 *Fasteners for Use in Structural Applications*

ASME B46.1-02 *Surface Texture, Surface Roughness, Waviness, and Lay*

ASTM International (ASTM)

A6/A6M-09 *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*

A36/A36M-08 *Standard Specification for Carbon Structural Steel*

A53/A53M-07 *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*

A193/A193M-08b *Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service*

A194/A194M-09 *Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High-Temperature Service, or Both*

A216/A216M-08 *Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High Temperature Service*

A242/A242M-04 *Standard Specification for High-Strength Low-Alloy Structural Steel*

A283/A283M-03 *Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates*

A307-07b *Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength*

A325-09 *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

A325M-09 *Standard Specification for High-Strength Bolts for Structural Steel Joints (Metric)*

A354-07a *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*

A370-09 *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

A449-07b *Standard Specification for Quenched and Tempered Steel Bolts and Studs*

A490-08b *Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*

A490M-08 *Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)*

A500-03a *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*

A501-07 *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*

A502-03 *Standard Specification for Steel Structural Rivets*

A514/A514M-05 *Standard Specification for High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding*

A529/A529M-05 *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*

A563-07a *Standard Specification for Carbon and Alloy Steel Nuts*

A563M-07 *Standard Specification for Carbon and Alloy Steel Nuts [Metric]*

A568/A568M-09 *Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for*

A572/A572M-07 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

A588/A588M-05 *Standard Specification for High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100 mm Thick*

A606/A606M-09 *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*

A618/A618M-04 *Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing*

A668/A668M-04 *Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use*

A673/A673M-04 *Standard Specification for Sampling Procedure for Impact Testing of Structural Steel*

A709/A709M-09 *Standard Specification for Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates, and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges*

A751-08 *Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products*

A847/A847M-05 *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*

A852/A852M-03(2007) *Standard Specification for Quenched and Tempered Low-Alloy Structural Steel Plate with 485 MPa Minimum Yield Strength to 100 mm Thick*

A913/A913M-07 *Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)*

A992/A992M-06a *Standard Specification for Steel for Structural Shapes for Use in Building Framing*

User Note: ASTM A992 is the most commonly referenced specification for W shapes.

A101 1/A101 1M-09a *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability*

C33-03 *Standard Specification for Concrete Aggregates*

C330-04 *Standard Specification for Lightweight Aggregates for Structural Concrete*

E119-08a *Standard Test Methods for Fire Tests of Building Construction and Materials*

E709-08 *Standard Guide for Magnetic Particle Examination*

F436-09 *Standard Specification for Hardened Steel Washers*

F959-09 *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*

F1554-07a *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength*

User Note: ASTM F1554 is the most commonly referenced specification for anchor rods. Grade and weldability must be specified.

F1852-08 *Standard Specification for "Twist-Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

American Welding Society (AWS)

AWS D1. 1/D 1. 1M-2004 *Structural Welding Code-Steel*

AWS A5.1/A5.1M-2004 *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.5/A5.5M-2004 *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.17/A5.17M-97 *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding*

AWS A5.18/A5.18M-2005 *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.20/A5.20M-2005 *Specification for Carbon Steel Electrodes for Flux Cored Arc Welding*

AWS A5.23/A5.23M-2007 *Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding*

AWS A5.25/A5.25M-97 (R2009) *Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electrode Gas Welding*

AWS A5.26/A5.26M-97 (R2009) *Specification for Carbon and Low-Alloy Steel Electrodes for Electrode Gas Welding*

AWS A5.28/A5.28M-2005 *Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.29/A5.29M-2005 *Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding*

Research Council on Structural Connections (RCSC)

Specification for Structural Joints Using ASTM A325 or A490 Bolts, 2004

501.3 Material**501.3.1 Structural Steel Materials**

Material test reports or report of tests made by the fabricator or a testing laboratory shall constitute sufficient evidence of conformity with one of the above listed ASTM standards. For hot-rolled structural shapes, plates, and bars, such tests shall be made in accordance with ASTM A6/A6M; for sheets, such tests shall be made in accordance with ASTM A568/A568M; for tubing and pipe, such tests shall be made in accordance with the requirements of the applicable ASTM standards listed above for those product forms. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

501.3.1.1 ASTM Designations

Structural steel material conforming to one of the following ASTM specifications is approved for use under this Specification:

1. Hot-rolled structural shapes:

ASTM A36 /A36M
 ASTM A529/ A529M
 ASTM A572/ A572M
 ASTM A588/ A588M
 ASTM A709/ A709M
 ASTM A913/ A913M
 ASTM A992/ A992M

2. Structural tubing:

ASTM A500
 ASTM A501
 ASTM A618
 ASTM A847

3. Pipe:

ASTM A53/A53M, Gr. B

4. Plates:

ASTM A36/A36M
 ASTM A242/A242M
 ASTM A283/A283M
 ASTM A514/A514M
 ASTM A529/A529M
 ASTM A572/A572M
 ASTM A588/A588M
 ASTM A709/A709M
 ASTM A852/A852M
 ASTM A1011/A1011M

5. Bars:

ASTM A36/A36M
 ASTM A529/A529M
 ASTM A572/A572M
 ASTM A709/A709M

6. Sheets:

ASTM A606
 A1011/A1011MSS
 HSLAS
 HSLAS-F

501.3.1.2 Unidentified Steel

Unidentified steel free of injurious defects is permitted to be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

501.3.1.3 Rolled Heavy Shapes

ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm, used as members subject to primary (computed) tensile forces due to tension or flexure and spliced using complete-joint-penetration groove welds that fuse through the thickness of the member, shall be specified as follows

The contract documents shall require that such shapes be supplied with Charpy V-Notch (CVN) impact test results in accordance with ASTM A6/A6M,

Supplementary Requirement S30, Charpy V-Notch Impact Test for Structural Shapes – Alternate Core Location. The impact test shall meet a minimum average value of 27 J absorbed energy at +21 °C.

The above requirements do not apply if the splices and connections are made by bolting. The above requirements do not apply to hot-rolled shapes with a flange thickness exceeding 50 mm that have shapes with flange or web elements less than 50 mm thick welded with complete-joint-penetration groove welds to the face of the shapes with thicker elements.

User Note: Additional requirements for joints in heavy rolled members are given in Sections 510.1.5, 510.1.6, 510.2.7, and 513.2.2.

501.3.1.4 Built-Up Heavy Shapes

Built-up cross-sections consisting of plates with a thickness exceeding 50 mm, used as members subject to primary (computed) tensile forces due to tension or flexure and spliced or connected to other members using complete joint-penetration groove welds that fuse through the thickness of the plates, shall be specified as follows. The contract documents shall require that the steel be supplied with Charpy V-Notch impact test results in accordance with ASTM A6/A6M, Supplementary Requirement S5, Charpy V-Notch Impact Test. The impact test shall be conducted in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 27 J absorbed energy at +21 °C.

The above requirements also apply to built-up cross-sections consisting of plates exceeding 50 mm that are welded with complete-joint-penetration groove welds to the face of other sections.

User Note: Additional requirements for joints in heavy built-up members are given in Sections 510.1.5, 510.1.6, 510.2.7 and 513.2.2.

501.3.2 Steel Castings and Forgings

Cast steel shall conform to ASTM A216/A216M, Gr. WCB with Supplementary Requirement S11. Steel forgings shall conform to ASTM A668/A668M. Test reports produced in accordance with the above reference standards shall constitute sufficient evidence of conformity with such standards.

501.3.3 Bolts, Washers and Nuts

Bolt, washer, and nut material conforming to one of the following ASTM specifications is approved for use under this Specification:

1. Bolts:

ASTM A307
ASTM A325/A325M
ASTM A449
ASTM A490/A490M
ASTM F1852

2. Nuts:

ASTM A194/A194M
ASTM A563/A563M

3. Washers:

ASTM F436/F436M

4. Compressible-Washer-Type Direct Tension Indicators:

ASTM F959/F959M

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

501.3.4 Anchor Rods and Threaded Rods

Anchor rod and threaded rod material conforming to one of the following ASTM specifications is approved for use under this Specification:

ASTM A36/A36M
ASTM A193/A193M
ASTM A354
ASTM A449
ASTM A572/A572M
ASTM A588/A588M
ASTM F1554

User Note: ASTM F1554 is the preferred material specification for anchor rods.

A449 material is acceptable for high-strength anchor rods and threaded rods of any diameter.

Threads on anchor rods and threaded rods shall conform to the Unified Standard Series of ASME B18.2.6 and shall have Class 2A tolerances.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

501.3.5 Consumables for Welding

Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society:

AWS A5.1
AWSA5.5
AWS A5.17/A5.17M
AWS A5.18
AWSA5.20
AWS A5.23/A5.23M
AWS A5.25/A5.25M
AWS A5.26/A5.26M
AWSA5.28
AWSA5.29
AWS A5.32/A5.32M

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards. Filler metals and fluxes that are suitable for the intended application shall be selected.

501.3.6 Headed Stud Anchors

Steel stud shear connectors shall conform to the requirements of Structural Welding Code-Steel, AWS D1.1.

User Note: Studs are made from cold drawn bar, either semi-killed or killed aluminum or silicon deoxidized, conforming to the requirements of ASTM A29/ A29M-04, Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for.

Manufacturer's certification shall constitute sufficient evidence of conformity with AWS D1.1.

501.4 Structural Design Drawings and Specifications

See Section 106.

SECTION 502 DESIGN REQUIREMENTS

502.1 General Provisions

The design of members and connections shall be consistent with the intended behavior of the framing system and the assumptions made in the structural analysis. Unless restricted by this chapter, lateral load resistance and stability may be provided by any combination of members and connections.

502.2 Loads and Load Combinations

The loads and load combinations shall be as stipulated by this chapter. In the absence of a building code, the loads and load combinations shall be those stipulated in Chapter 2. For design purposes, the nominal loads shall be taken as the loads stipulated by this chapter.

502.3 Design Basis

Designs shall be made according to the provisions for Load and Resistance Factor Design (LRFD) or to the provisions for Allowable Strength Design (ASD).

502.3.1 Required Strength

The required strength of structural members and connections shall be determined by structural analysis for the appropriate load combinations as stipulated in Section 502.2.

Design by elastic, inelastic or plastic analysis is permitted. Provisions for inelastic and plastic analysis are as stipulated in Appendix A-1, *Inelastic Analysis and Design*. The provisions for moment redistribution in continuous beams in Appendix A-1, Section A-1.3 are permitted for elastic analysis only.

502.3.2 Limit States

Design shall be based on the principle that no applicable strength or serviceability limit state shall be exceeded when the structure is subjected to all appropriate load combinations.

502.3.3 Design for Strength Using Load and Resistance Factor Design (LRFD)

Design according to the provisions for LRFD satisfies the requirements of this chapter when the design strength of each structural component equals or exceeds the required strength determined on the basis of the LRFD load combinations. All provisions of this chapter, except for those in Section 502.3.4, shall apply.

Design shall be in accordance with Eq. 502.3- 1:

$$R_u \leq \phi R_n \quad (502.3-1)$$

where

- R_u = required strength (LRFD)
- R_n = nominal strength, specified in Sections 502 through 511
- ϕ = resistance factor, specified in Sections 502 through 511
- ϕR_n = design strength

502.3.4 Design for Strength Using Allowable Strength Design (ASD)

Design according to the provisions for ASD satisfies the requirements of this chapter when the allowable strength of each structural component equals or exceeds the required strength determined on the basis of the ASD load combinations. All provisions of this chapter, except those of Section 502.3.3, shall apply.

Design shall be in accordance with Eq. 502. 3-2:

$$R_a \leq R_n / \Omega \quad (502.3-2)$$

where

- R_a = required strength (ASD)
- R_n = nominal strength, specified in Sections 502 through 511
- Ω = safety factor, specified in Sections 502 through 511
- R_n / Ω = allowable strength

502.3.5 Design for Stability

Stability of the structure and its elements shall be determined in accordance with Section 503.

502.3.6 Design of Connections

Connection elements shall be designed in accordance with the provisions of Sections 510 and 511. The forces and deformations used in design shall be consistent with the intended performance of the connection and the assumptions used in the structural analysis.

502.3.6.1 Simple Connections

A simple connection transmits a negligible moment across the connection. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure. Inelastic rotation of the connection is permitted.

502.3.6.2 Moment Connections

A moment connection transmits moment across the connection. Two types of moment connections, FR and PR, are permitted, as specified below.

1. Fully-Restrained (FR) Moment Connections

Fully-restrained (FR) moment connections transfer moment with a negligible rotation between the connected members. In the analysis of the structure, the connection may be assumed to allow no relative rotation. An FR connection shall have sufficient strength and stiffness to maintain the angle between the connected members at the strength limit states.

2. Partially-Restrained (PR) Moment Connections

Partially-restrained (PR) moment connections transfer moments, but the rotation between connected members is not negligible. In the analysis of the structure, the force-deformation response characteristics of the connection shall be included. The response characteristics of a PR connection shall be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection shall have sufficient strength, stiffness, and deformation capacity at the strength limit states.

502.3.9 Design for Serviceability

The overall structure and the individual members, connections, and connectors shall be checked for serviceability. Performance requirements for serviceability design are given in Section 512.

502.3.10 Design for Ponding

The roof system shall be investigated through structural analysis to assure adequate strength and stability under ponding conditions, unless the roof surface is provided with a slope of 20 mm per meter or greater toward points of free drainage or an adequate system of drainage is provided to prevent the accumulation of water.

See Appendix A-2, *Design for Ponding*, for methods of checking ponding.

502.3.11 Design for Fatigue

Fatigue shall be considered in accordance with Appendix A-3, *Design for Fatigue*, for members and their connections subject to repeated loading. Fatigue need not be considered for seismic effects or for the effects of wind loading on normal building lateral load resisting systems and building enclosure components.

502.3.12 Design for Fire Conditions

Two methods of design for fire conditions are provided in Appendix A-4, *Structural Design for Fire Conditions: Qualification Testing and Engineering Analysis*. Compliance with the fire protection requirements in this Chapter shall be deemed to satisfy the requirements of this section and Appendix A-4.

Nothing in this section is intended to create or imply a contractual requirement for the engineer-of-record responsible for the structural design or any other member of the design team.

502.3.13 Design for Corrosion Effects

Where corrosion may impair the strength or serviceability of a structure, structural components shall be designed to tolerate corrosion or shall be protected against corrosion.

502.4 Member Properties

502.4.1 Classification of Sections for Local Buckling

Sections are classified as compact, non-compact, or slender-element sections. For a section to qualify as compact its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios λ_p from Tables 502.4.1 and 502.4.2. If the width-thickness ratio of one or more compression elements exceeds λ_p , but does not exceed λ_r from Tables 502.4.1 and 502.4.2, the section is noncompact. If the width-thickness ratio of any element exceeds λ_r , the section is referred to as a slender-element section.

502.4.2 Unstiffened Elements

For unstiffened elements supported along only one edge parallel to the direction of the compression force, the width shall be taken as follows:

1. For flanges of I-shaped members and tees, the width b is one-half the full-flange width, b_f .
2. For legs of angles and flanges of channels and zees, the width b is the full nominal dimension.
3. For plates, the width b is the distance from the free edge to the first row of fasteners or line of welds.
4. For stems of tees, d is taken as the full nominal depth of the section.

User Note: Refer to Table 502.4.1 for the graphic representation of unstiffened element dimensions.

502.4.3 Stiffened Elements

For stiffened elements supported along two edges parallel to the direction of the compression force, the width shall be taken as follows:

1. For webs of rolled or formed sections, h is the clear distance between flanges less the fillet or corner radius at each flange; h_c is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.
2. For webs of built-up sections, h is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and h_c is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the

inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.

3. For flange or diaphragm plates in built-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.
4. For flanges of rectangular hollow structural sections (HSS), the width b is the clear distance between web less the inside corner radius on each side. For webs of rectangular HSS, h is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, b and h shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, t , shall be taken as the design wall thickness, per Section 502.3.12.

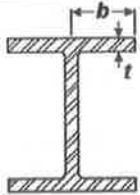
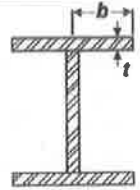
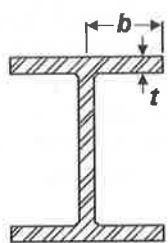
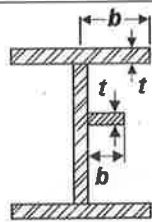
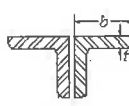
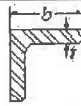
User Note: Refer to Table 502.4.2 for the graphic representation of stiffened element dimensions.

For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

503 .1 Design Wall Thickness for HSS

The design wall thickness, t , shall be used in calculations involving the wall thickness of hollow structural sections (HSS). The design wall thickness, t , shall be taken equal to 0.93 times the nominal wall thickness for electric-resistance welded (ERW) HSS and equal to the nominal thickness for submerged-arc welded (SAW) HSS.

Table 502.4.1 Limiting Width-Thickness Ratios for Compression Elements

Unstiffened Elements	Case	Description of Elements	Width-Thickness Ratio	Limiting Width-Thickness Ratios		Example
				λ_p (compact)	λ_r (non-compact)	
	1	Flexure in flanges of rolled I-shaped sections and channels	b/t	$0.38 \sqrt{E/F_y}$	$1.0 \sqrt{E/F_y}$	
	2	Flexure in flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38 \sqrt{E/F_y}$	$0.95 \sqrt{k_c E/F_L}^{[a][b]}$	
	3	Uniform compression in flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles in continuous contact and flanges of channels	b/t	NA	$0.56 \sqrt{E/F_y}$	
	4	Uniform compression in flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	NA	$0.64 \sqrt{k_c E/F_y}^{[a]}$	
	5	Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	NA	$0.45 \sqrt{E/F_y}$	
	6	Flexure in legs of single angles	b/t	$0.54 \sqrt{E/F_y}$	$0.91 \sqrt{E/F_y}$	

^[a] $k_c = \frac{4}{\sqrt{h/t_w}}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes. (See Cases 2 and 4)

^[b] $F_L = 0.7F_y$ for minor-axis bending, major axis bending of slender-web built-up I-shaped members, and major axis bending of compact and non-compact web built-up I-shaped members with $S_{xt}/S_{xc} \geq 0.7$; $F_L = F_y S_{xt}/S_{xc} \geq 0.5F_y$ for major-axis bending of compact and non-compact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$. (See Case 2)

Table 502.4.2 Limiting Width-Thickness Ratios for Compression Elements

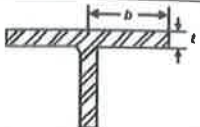
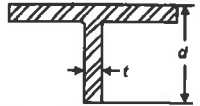
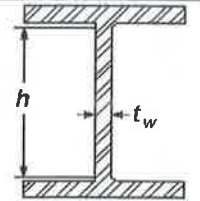
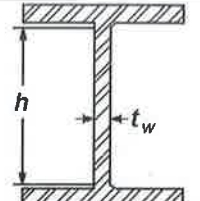
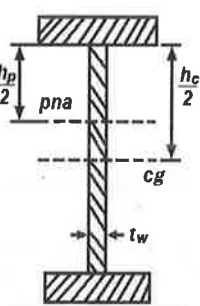
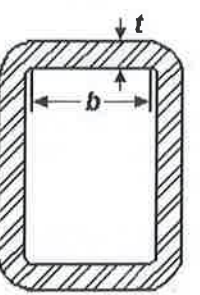
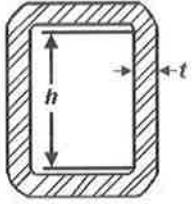
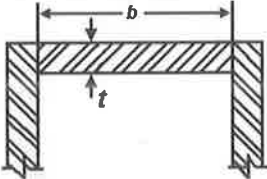
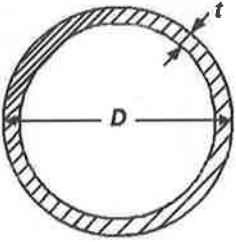
	Case	Description of Elements	Width-Thickness Ratio	Limiting Width-Thickness Ratios		Example
				λ_p (compact)	λ_r (non-compact)	
Stiffened Elements	7	Flexure in flanges of tees	b/t	$0.38 \sqrt{E/F_y}$	$1.0 \sqrt{E/F_y}$	
	8	Uniform compression in stems of tees	d/t_w	NA	$0.75 \sqrt{E/F_y}$	
	9	Flexure in webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76 \sqrt{E/F_y}$	$5.70 \sqrt{E/F_y}$	
	10	Uniform compression in webs of doubly symmetric I-shaped sections	h/t_w	NA	$1.49 \sqrt{E/F_y}$	
	11	Flexure in webs of singly-symmetric I-shaped sections	h_c/t_w	$\frac{\frac{h_c}{h_p} \sqrt{\frac{E}{F_y}}}{\left(0.54 \frac{M_p}{M_y} - 0.09\right)^2} \leq \lambda_r$	$5.70 \sqrt{E/F_y}$	
	12	Uniform compression in flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12 \sqrt{E/F_y}$	$1.40 \sqrt{E/F_y}$	

Table 502.4.2 Limiting Width-Thickness Ratios for Compression Elements (continuation)

	Case	Description of Elements	Width-Thickness Ratio	Limiting Width-Thickness Ratios		Example
				λ_p (compact)	λ_r (non-compact)	
Stiffened Elements	13	Flexure in webs of rectangular HSS	h/t	$2.42 \sqrt{E/F_y}$	$5.70 \sqrt{E/F_y}$	
	14	Uniform compression in all other stiffened elements	b/t	NA	$1.49 \sqrt{E/F_y}$	
	15	Circular hollow sections In uniform compression In Flexure	D/t D/t	NA $0.07E/F_y$	$0.11E/F_y$ $0.31E/F_y$	

502.5 Fabrication, Erection and Quality Control

Shop drawings, fabrication, shop painting, erection, and quality control shall meet the requirements stipulated in Section 513, Fabrication, Erection, and Quality Control.

502.6 Evaluation of Existing Structures

Provisions for the evaluation of existing structures are presented in Appendix A-5, *Evaluation of Existing Structures*.

SECTION 503

STABILITY ANALYSIS AND DESIGN

503.1 Stability Design Requirements

503.1.1 General Stability Requirements

Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of second-order effects (including $P - \Delta$ and $P - \delta$ effects), flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due to residual stresses on the stability of the structure and its elements is permitted. The methods prescribed in this section and Appendix A-7, *Direct Analysis Method*, satisfy these requirements. All component and connection deformations that contribute to the lateral displacements shall be considered in the stability analysis.

In structures designed by elastic analysis, individual member stability and stability of the structure as a whole are provided jointly by:

1. Calculation of the required strengths for members, connections and other elements using one of the methods specified in Section 503.2.2, and
2. Satisfaction of the member and connection design requirements in this specification based upon those required strengths.

In structures designed by inelastic analysis, the provisions of Appendix A-1, *Inelastic Analysis and Design*, shall be satisfied.

503.1.2 Member Stability Design Requirements

Individual member stability is provided by satisfying the provisions of Sections 505, 506, 507, 508 and 509.

User Note: Local buckling of cross section components can be avoided by the use of compact sections defined in Section 502.4.

Where elements are designed to function as braces to define the unbraced length of columns and beams, the bracing system shall have sufficient stiffness and strength to control member movement at the braced points. Methods of satisfying this requirement are provided in Appendix A-6, *Stability Bracing for Columns and Beams*.

503.1.3 System Stability Design Requirements

Lateral stability shall be provided by moment frames, braced frames, shear walls, and/or other equivalent lateral load resisting systems. The overturning effects of drift and the destabilizing influence of gravity loads shall be considered. Force transfer and load sharing between elements of the framing systems shall be considered. Braced-frame and shear-wall systems, moment frames, gravity framing systems, and combined systems shall satisfy the following specific requirements:

503.1.3.1 Braced-Frame and Shear-Wall Systems

In structures where lateral stability is provided solely by diagonal bracing, shear walls, or equivalent means, the effective length factor, K , for compression members shall be taken as 1.0, unless structural analysis indicates that a smaller value is appropriate. In braced-frame systems, it is permitted to design the columns, beams, and diagonal members as a vertically cantilevered, simply connected truss.

User Note: Knee-braced frames function as moment-frame systems and should be treated as indicated in Section 503.1.3.2. Eccentrically braced frame systems function as combined systems and should be treated as indicated in Section 503.1.3.4.

503.1.3.2 Moment-Frame Systems

In frames where lateral stability is provided by the flexural stiffness of connected beams and columns, the effective length factor K or elastic critical buckling stress, F_e , for columns and beam-columns shall be determined as specified in Section 503.2.

503.1.3.3 Gravity Framing Systems

Columns in gravity framing systems shall be designed based on their actual length ($K = 1.0$) unless analysis shows that a smaller value may be used. The lateral stability of gravity framing systems shall be provided by moment frames, braced frames, shear walls, and/or other equivalent lateral load resisting systems. $P - \Delta$ effects due to load on the gravity columns shall be transferred to the lateral load resisting systems and shall be considered in the calculation of the required strengths of the lateral load resisting systems.

503.1.3.4 Combined Systems

The analysis and design of members, connections and other elements in combined systems of moment frames, braced frames, and/or shear walls and gravity frames shall meet the requirements of their respective systems.

503.2 Calculation of Required Strengths

Except as permitted in Section 503.2.2.2, required strengths shall be determined using a second-order analysis as specified in Section 503.2.1. Design by either second-order or first-order analysis shall meet the requirements specified in Section 503.2.2.

503.2.1 Methods of Second-Order Analysis

Second-order analysis shall conform to the requirements of this Section.

503.2.1.1 General Second-Order Elastic Analysis

Any second-order elastic analysis method that considers both $P - \Delta$ and $P - \delta$ effects may be used.

The amplified First-Order Elastic Analysis Method defined in Section 503.2.1.2 is an accepted method for second-order elastic analysis of braced, moment, and combined framing systems.

503.2.1.2 Second-Order Analysis by Amplified First-Order Elastic Analysis

User Note: A method is provided in this section to account for second-order effects in frames by amplifying the axial forces and moments in members and connections from a first-order analysis.

The following is an approximate second-order analysis procedure for calculating the required flexural and axial strengths in members of lateral load resisting systems. The required second-order flexural strength, M_r , and axial strength, P_r , shall be determined as follows:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \quad (503.2-1a)$$

$$P_r = P_{nt} + B_2 P_{lt} \quad (503.2-1b)$$

where

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \quad (503.2-2)$$

For members subjected to axial compression, B_1 may be calculated based on the first-order estimate $P_r = P_{nt} + P_{lt}$.

User Note: B_1 is an amplifier to account for second order effects caused by displacements between brace points ($P - \delta$) and B_2 is an amplifier to account for second order effects caused by displacements of braced points ($P - \Delta$).

For members in which $B_1 \leq 1.05$, it is conservative to amplify the sum of the non-sway and sway moments (as obtained, for instance, by a first-order elastic analysis) by the B_2 amplifier, in other words, $M_r = B_2(M_{nt} + M_{lt})$.

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}} \geq 1 \quad (503.2-3)$$

User Note: Note that the B_2 amplifier (Eq. 503.2-3) can be estimated in preliminary design by using a maximum lateral drift limit corresponding to the story shear $\sum H$ in Eq. 503.2-6b.

and

$$\alpha = 1.00(LRFD) \quad \alpha = 1.60(ASD)$$

- M_r = required second-order flexural strength using LRFD or ASD load combinations, N-mm
- M_{nt} = first-order moment using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, N-mm
- M_{lt} = first-order moment using LRFD or ASD load combinations caused by lateral translation of the frame only, N-mm
- P_r = required second-order axial strength using LRFD or ASD load combinations, N
- P_{nt} = first-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, N
- $\sum P_{nt}$ = total vertical load supported by the story using LRFD or ASD load combinations, including gravity column loads, N
- P_{lt} = first-order axial force using LRFD or ASD load combinations caused by lateral translation of the frame only, N
- C_m = is a coefficient assuming no lateral translation of the frame whose value shall be taken as follows:

- a. For beam-columns not subject to transverse loading between supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1/M_2) \quad (503.2-4)$$

Where M_1 and M_2 , calculated from a first-order analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature.

- b. For beam-columns subjected to transverse loading between supports, the value of C_m shall be determined either by analysis or conservatively taken as 1.0 for all cases.

P_{e1} = elastic critical buckling resistance of the member in the plane of bending, calculated based on the assumption of zero sidesway, N

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \quad (503.2-5)$$

ΣP_{e2} = elastic critical buckling resistance for the story determined by sidesway buckling analysis, N

For moment frames, where sidesway buckling effective length factors K_2 are determined for the columns, it is permitted to calculate the elastic story sidesway buckling resistance as

$$\Sigma P_{e2} = \sum \frac{\pi^2 EI}{(K_2 L)^2} \quad (503.2-6a)$$

For all types of lateral load resisting systems, it is permitted to use

$$\Sigma P_{e2} = R_M \frac{\Sigma H L}{\Delta_H} \quad (503.2-6b)$$

where

E = modulus of elasticity of steel = 200,000 MPa

R_M = 1.0 for braced-frame systems;
= 0.85 for moment-frame and combined systems, unless a larger value is justified by analysis

I = moment of inertia in the plane of bending, mm⁴

L = story height, mm

K_1 = effective length factor in the plane of bending, calculated based on the assumption of no lateral translation, set equal to 1.0 unless analysis indicates that a smaller value may be used

K_2 = effective length factor in the plane of bending, calculated based on a sidesway buckling analysis

User Note: Methods for calculation of K_2 are discussed in the AISC Commentary.

Δ_H = first-order interstory drift due to lateral forces, mm. Where Δ_H varies over the plan area of the structure, Δ_H shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift

ΣH = story shear produced by the lateral forces used to compute Δ_H , N

503.2.2 Design Requirements

These requirements apply to all types of braced, moment, and combined framing systems. Where the ratio of second-order drift to first-order drift is equal to or less than 1.5, the required strengths of members, connections and other elements shall be determined by one of the methods specified in Sections 503.2.2.1 or 503.2.2.2, or by the *Direct Analysis Method* of Appendix A-7. Where the ratio of second-order drift to first-order drift is greater than 1.5, the required strengths shall be determined by the *Direct Analysis Method* of Appendix A-7.

User Note: The ratio of second-order drift to first-order drift can be represented by B_2 , as calculated using Eq. 503.2-3. Alternatively, the ratio can be calculated by comparing the results of a second-order analysis to the results of a first-order analysis, where the analyses are conducted either under LRFD load combinations directly or under ASD load combinations with a 1.6 factor applied to the ASD gravity loads.

For the methods specified in Sections 503.2.2.1 or 503.2.2.2:

1. Analysis shall be conducted according to the design and loading requirements specified in either Section 502.3.3 (LRFD) or Section 502.3.4 (ASD).

- The structure shall be analyzed using the nominal geometry and the nominal elastic stiffness for all elements.

503.2.2.1 Design by Second-Order Analysis

Where required strengths are determined by a second-order analysis:

- The provisions of Section 503.2.1 shall be satisfied.
- For design by ASD, analyses shall be carried out under 1.6 times the ASD load combinations and the results shall be divided by 1.6 to obtain the required strengths.

User Note: The amplified first order analysis method of Section 503.2.1.2 incorporates the 1.6 multiplier directly in the B_1 and B_2 amplifiers, such that no other modification is needed.

- All gravity-only load combinations shall include a minimum lateral load applied at each level of the structure of $0.002Y_i$, where Y_i is the design gravity load applied at level i , N. This minimum lateral load shall be considered independently in two orthogonal directions.

User Note: The minimum lateral load of $0.002Y_i$, in conjunction with the other design-analysis constraints listed in this section, limits the error that would otherwise be caused by neglecting initial out-of-plumbness and member stiffness reduction due to residual stresses in the analysis.

- Where the ratio of second-order drift to first-order drift is less than or equal to 1.1, members are permitted to be designed using $K = 1.0$. Otherwise, columns and beam-columns in moment frames shall be designed using a K factor or column buckling stress, F_e , determined from a sidesway buckling analysis of the structure. Stiffness reduction adjustment due to column inelasticity is permitted in the determination of the K factor. For braced frames, K for compression members shall be taken as 1.0, unless structural analysis indicates a smaller value may be used.

503.2.2.2 Design by First-Order Analysis

Required strengths are permitted to be determined by a first-order analysis, with all members designed using $K = 1.0$, provided that

- The required compressive strengths of all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure satisfy the following limitation:

$$\alpha P_r \leq 0.5 P_y \quad (503.2-7)$$

where

$$\alpha = 1.0 \text{ (LRFD)} \quad \alpha = 1.6 \text{ (ASD)}$$

P_r = required axial compressive strength under LRFD or ASD load combinations, N

P_y = member yield strength ($= AF_y$), N

- All load combinations include an additional lateral load, N_i , applied in combination with other loads at each level of the structure, where

$$N_i = 2.1(\Delta/L)Y_i \geq 0.0042Y_i \quad (503.2-8)$$

where

Y_i = gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i , N

Δ/L = the maximum ratio of Δ to L for all stories in the structure

Δ = first-order interstory drift due to the design loads, mm. Where Δ varies over the plan area of the structure, Δ shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift

L = story height, mm

User Note: The drift Δ is calculated under LRFD load combinations directly or under ASD load combinations with a 1.6 factor applied to the ASD gravity loads.

This additional lateral load shall be considered independently in two orthogonal directions.

- The non-sway amplification of beam-column moments is considered by applying the B_1 amplifier of Section 503.2.1 to the total member moments.

SECTION 504

DESIGN OF MEMBERS FOR TENSION

504.1 Slenderness Limitations

There is no maximum slenderness limit for design of members in tension.

User Note: For members designed on the basis of tension, the slenderness ratio L/r preferably should not exceed 300. This suggestion does not apply to rods or hangers in tension.

504.2 Tensile Strength

The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_t , of tension members, shall be the lower value obtained according to the limit states of tensile yielding in the gross section and tensile rupture in the net section.

1. For tensile yielding in the gross section:

$$P_n = F_y A_g \quad (504.2-1)$$

$$\phi_t = 0.90 \text{ (LRFD)} \quad \Omega_t = 1.67 \text{ (ASD)}$$

2. For tensile rupture in the net section:

$$P_n = F_u A_e \quad (504.2-2)$$

$$\phi_t = 0.75 \text{ (LRFD)} \quad \Omega_t = 2.00 \text{ (ASD)}$$

where

A_e = effective net area, mm²

A_g = gross area of member, mm²

F_y = specified minimum yield stress of the type of steel being used, MPa

F_u = specified minimum tensile strength of the type of steel being used, MPa

When members without holes are fully connected by welds, the effective net area used in Eq. 504.2-2 shall be as defined in Section 504.3. When holes are present in a member with welded end connections, or at the welded connection in the case of plug or slot welds, the effective net area through the holes shall be used in Equation 504.2-2.

504.3 Area Determination

504.3.1 Gross Area

The gross area, A_g , of a member is the total cross-sectional area.

504.3.2 Net Area

The net area, A_n , of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken 2 mm greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section 510.3.2, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/4g$.

where

s = longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm

g = transverse center-to-center spacing (gage) between fastener gage lines, mm

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

For slotted HSS welded to a gusset plate, the net area, A_n , is the gross area minus the product of the thickness and the total width of material that is removed to form the slot. In determining the net area across plug or slot welds, the weld metal shall not be considered as adding to the net area.

User Note: Section 510.4.1.(2) limits A_n to a maximum of $0.85A_g$ connection design for splice plates with holes.

504.3.3 Effective Net Area

The effective area of tension members shall be determined as follows:

$$A_e = A_n U \quad (504.3-1)$$

where U , the shear lag factor, is determined as shown in Table 504.3. 1.

Members such as single angles, double angles and **WT** sections shall have connections proportioned such that **U** is equal to or greater than 0.60. Alternatively, a lesser value of **U** is permitted if these tension members are designed for the effect of eccentricity in accordance with Section 508.1.2 or Section 508.2.

504.4 Built-up Members

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section 510.3.5.

Either perforated cover plates or tie plates without lacing are permitted to be used on the open sides of built-up tension members. Tie plates shall have a length not less than two-thirds the distance between the lines of welds or fasteners connecting them to the components of the member. The thickness of such tie plates shall not be less than one-fiftieth of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed 150 mm.

User Note: The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300.

504.5 Pin-Connected Members

504.5.1 Tensile Strength

The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_t of pin-connected members, shall be the lower value obtained according to the limit states of tensile rupture, shear rupture, bearing, and yielding.

1. For tensile rupture on the net effective area:

$$P_n = 2tb_{eff}F_u \quad (504.5-1)$$

$$\phi_t = 0.75 \text{ (LRFD)} \quad \Omega_t = 2.00 \text{ (ASD)}$$

2. For shear rupture on the effective area:

$$P_n = 0.6F_u A_{sf} \quad (504.5-2)$$

$$\phi_{sf} = 0.75 \text{ (LRFD)} \quad \Omega_{sf} = 2.00 \text{ (ASD)}$$

where

$$A_{sf} = 2t(a + d/2), \text{ mm}^2$$

a = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the force, mm

b_{eff} = **2t + 16**, mm but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force

d = pin diameter, mm

t = thickness of plate, mm

3. For bearing on the projected area of the pin, see Section 510.7.
4. For yielding on the gross section, use Eq. 504.2-1.

504.5.2 Dimensional Requirements

The pin hole shall be located midway between the edges of the member in the direction normal to the applied force. When the pin is expected to provide for relative movement between connected parts while under full load, the diameter of the pin hole shall not be more than 1 mm greater than the diameter of the pin.

The width of the plate at the pin hole shall not be less than **2b_{eff} + d** and the minimum extension, **a**, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than **1.33b_{eff}**.

The corners beyond the pin hole are permitted to be cut at 45° to the axis of the member, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

504.6 Eyebars

504.6.1 Tensile Strength

The available tensile strength of eyebars shall be determined in accordance with Section 504.2, with **A_g** taken as the cross-sectional area of the body.

For calculation purposes, the width of the body of the eyebars shall not exceed eight times its thickness.

504.6.2 Dimensional Requirements

Eyebars shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads with the periphery concentric with the pin hole.

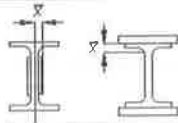
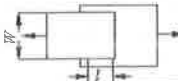
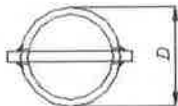
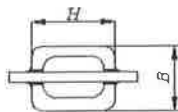
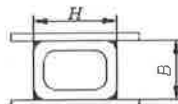
The radius of transition between the circular head and the eyebar body shall not be less than the head diameter.

The pin diameter shall not be less than seven-eighths times the eyebar body width, and the pin hole diameter shall not be more than 1 mm greater than the pin diameter.

For steels having F_y greater than 485 MPa, the hole diameter shall not exceed five times the plate thickness, and the width of the eyebar body shall be reduced accordingly.

A thickness of less than 13 mm is permissible only if external nuts are provided to tighten pin plates and filler plates into snug contact. The width from the hole edge to the plate edge perpendicular to the direction of applied load shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

Table 504.3.1 Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds. (except as in Cases 3, 4, 5 and 6)		$U = 1.0$	
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds (Alternately, for W, M, S and HP, Case 7 may be used.)		$U = 1 - \frac{\bar{x}}{l}$	
3	All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \dots U = 1.0$ $2w > l \geq 1.5w \dots U = 0.87$ $1.5w > l \geq w \dots U = 0.75$	
5	Round HSS with a single concentric gusset plate.		$l \geq 1.3D \dots U = 1.0$ $D \leq l < 1.3D \dots U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \dots U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{B^2 + 2BH}{4(B + H)}$	
		with two side gusset plates	$l \geq H \dots U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{B^2}{4(B + H)}$	
7	W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq 2/3d \dots U = 0.90$ $b_f < 2/3d \dots U = 0.85$	
		with web connected with 4 or more fasteners per line in direction of loading	$U = 0.70$	
8	Single angles (If U is calculated per Case 2, the larger value is permitted to be used)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	
		with 2 or 3 fasteners per line in the direction of loading	$U = 0.60$	
l = length of connection, mm; w = plate width, mm; $\bar{x}$ = connection eccentricity, mm B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, mm; H = overall height of rectangular HSS member, measured in the plane of the connection, mm.				

SECTION 505

DESIGN OF MEMBERS FOR COMPRESSION

505.1 General Provisions

The design compressive strength, $\phi_c P_n$, and the allowable compressive strength, P_n/Ω_c , are determined as follows:

The nominal compressive strength, P_n , shall be the lowest value obtained according to the limit states of flexural buckling, torsional buckling and flexural-torsional buckling.

1. For doubly symmetric and singly symmetric members the limit state of flexural buckling is applicable.
2. For singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns, the limit states of torsional or flexural-torsional buckling are also applicable.

$$\phi_c = 0.90 \text{ (LRFD)} \quad \Omega_c = 1.67 \text{ (ASD)}$$

505.2 Slenderness Limitations and Effective Length

The effective length factor, K , for calculation of column slenderness, KL/r , shall be determined in accordance with Section 503,

where

- L = laterally unbraced length of the member, mm
 r = governing radius of gyration, mm
 K = the effective length factor, as shown below

User Note: For members designed on the basis of compression, the slenderness ratio KL/r preferably should not exceed 200.

505.3 Compressive Strength for Flexural Buckling of Members Without Slender Elements

This section applies to compression members with compact and noncompact sections, as defined in Section 502.4, for uniformly compressed elements.

User Note: When the torsional unbraced length is larger than the lateral unbraced length, then Section 505.4 may control the design of wide flange and similarly shaped columns.

The nominal compressive strength, P_n , shall be determined based on the limit state of flexural buckling.

$$P_n = F_{cr} A_g \quad (505.3-1)$$

The flexural buckling stress, F_{cr} , is determined as follows:

$$1. \text{ when } \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}} \quad (\text{or } F_e \geq 0.44F_y)$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}} \right] F_y \quad (505.3-2)$$

$$2. \text{ when } \frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}} \quad (\text{or } F_e < 0.44F_y)$$

$$F_{cr} = 0.877F_e \quad (505.3-3)$$

where

F_e = elastic critical buckling stress determined according to Eq. 505.3-4, Section 505.4, or the provisions of Section 503.2, as applicable, MPa.

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r} \right)^2} \quad (505.3-4)$$

User Note: The two equations for calculating the limits and applicability of Section 505.3, one based on KL/r and one based on F_e , provide the same result.

505.4 Compressive Strength for Torsional and Flexural-Torsional Buckling of Members without Slender Elements

This section applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns with compact and noncompact sections, as defined in Section 502.4 for uniformly compressed elements. These provisions are not required for single angles, which are covered in Section 505.5.

The nominal compressive strength, P_n , shall be determined based on the limit states of flexural-torsional and torsional buckling, as follows:

$$P_n = F_{cr} A_g \quad (505.4-1)$$

1. For double-angle and tee-shaped compression members:

$$F_{cr} = \left(\frac{F_{cry} + F_{crz}}{2H} \right) \left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}} \right] \quad (505.4-2)$$

where F_{cry} taken as F_{cr} from Eq. 505.3-2 or 505.3-3, for flexural buckling about the y-axis of symmetry and

$$\frac{KL}{r} = \frac{KL}{r_y}, \text{ and}$$

$$F_{crz} = \frac{GJ}{A_g \bar{r}_o^2} \quad (505.4-3)$$

2. For all other cases, F_{cr} shall be determined according to Eq. 505.3-2 or 505.3-3, using the torsional or flexural-torsional elastic buckling stress, F_e , determined as follows:

- a. For doubly symmetric members:

$$F_e = \left[\frac{\pi^2 EC_w}{(K_z L)^2} + GJ \right] \frac{1}{I_x + I_y} \quad (505.4-4)$$

- b. For singly symmetric members where y is the axis of symmetry:

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H} \right) \left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}} \right] \quad (505.4-5)$$

- c. For unsymmetric members, F_e is the lowest root of the cubic equation:

$$\begin{aligned} (F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) \\ - F_e^2(F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 \\ - F_e^2(F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \end{aligned} \quad (505.4-6)$$

where

A_g = gross area of member, mm²

C_w = warping constant, mm⁶

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \quad (505.4-7)$$

$$H = 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \quad (505.4-8)$$

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L}{r_x} \right)^2} \quad (505.4-9)$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y} \right)^2} \quad (505.4-10)$$

$$F_{ez} = \left(\frac{\pi^2 EC_w}{(K_z L)^2} + GJ \right) \frac{1}{A_g \bar{r}_o^2} \quad (505.4-11)$$

G = shear modulus of elasticity of steel

= 77 200 MPa

I_x, I_y = moment of inertia about the principal axes, mm⁴

J = torsional constant, mm⁴

K_z = effective length factor for torsional buckling

x_o, y_o = coordinates of shear center with respect to the centroid, mm

$\bar{r}_o$ = polar radius of gyration about the shear center, mm

r_y = radius of gyration about y-axis, mm

User Note: For doubly symmetric I-shaped sections, C_w may be taken as $I_y h_o^2/4$, where h_o is the distance between flange centroids, in lieu of a more precise analysis. For tees and double angles, omit term with C_w when computing F_{ez} and take x_o as 0.

505.5 Single Angle Compression Members

The nominal compressive strength, P_n , of single angle members shall be determined in accordance with Section 505.3 or Section 505.7, as appropriate, for axially loaded members, as well as those subject to the slenderness modification of Sections 505.5(1) or 505.5(2), provided the members meet the criteria imposed.

The effects of eccentricity on single angle members are permitted to be neglected when the members are evaluated as axially loaded compression members using one of the effective slenderness ratios specified below, provided that: (1) members are loaded at the ends in compression through the same one leg; (2) members are attached by welding or by minimum two-bolt connections; and (3) there are no intermediate transverse loads.

1. For equal-leg angles or unequal-leg angles connected through the longer leg that are individual members or are web members of planar trusses with adjacent web members attached to the same side of the gusset plate or chord:

- a. When $0 \leq \frac{L}{r_x} < 80$:

$$\frac{KL}{r} = 72 + 0.75 \frac{L}{r_x} \quad (505.5-1)$$

- b. When $\frac{L}{r_x} > 80$:

$$\frac{KL}{r} = 32 + 1.25 \frac{L}{r_x} \leq 200 \quad (505.5-2)$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, KL/r from Eqs. 505.5.1 and 505.5.2 shall be increased by adding $4[(b_l/b_s)^2 - 1]$, but KL/r of the members shall not be less than $0.95L/r_z$.

2. For equal-leg angles or unequal-leg angles connected through the longer leg that are web members of box or space trusses with adjacent web members attached to the same side of the gusset plate or chord:

- a. When $0 \leq \frac{L}{r_x} \leq 75$:

$$\frac{KL}{r} = 60 + 0.8 \frac{L}{r_x} \quad (505.5-3)$$

- b. When $\frac{L}{r_x} > 75$:

$$\frac{KL}{r} = 45 + \frac{L}{r_x} \leq 200 \quad (505.5-4)$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, KL/r from Eqs. 505.5.3 and 505.5.4 shall be increased by adding $6[(b_l/b_s)^2 - 1]$, but KL/r of the member shall not be less than $0.82L/r_z$,

where

L = length of member between work points at truss chord centerlines, mm

b_l = longer leg of angle, mm

b_s = shorter leg of angle, mm

r_x = radius of gyration about geometric axis parallel to connected leg, mm

r_z = radius of gyration for the minor principal axis, mm

3. Single-angle members with different end conditions from those described in Section 505.5(1) or 505.5(2), with leg length ratios greater than 1.7, or with transverse loading shall be evaluated for combined axial load and flexure using the provisions of Section 508. End connection to different legs on each end or to both legs, the use of single bolts or the attachment of adjacent web members to opposite sides of the gusset plate or chord shall constitute different end conditions requiring the use of Section 508 provisions.

505.6 Built-up Members

505.6.1 Compressive Strength

1. The nominal compressive strength of built-up members composed of two or more shapes that are interconnected by bolts or welds shall be determined in accordance with Sections 505.3, 505.4, or 505.7 subject to the following modification. In lieu of more accurate analysis, if the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, KL/r is replaced by $(KL/r)_m$ determined as follows:

- a. For intermediate connectors that are snug-tight bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_t}\right)^2} \quad (505.6-1)$$

- b. For intermediate connectors that are welded or pretensioned bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + 0.82 \frac{\alpha^2}{(1 + \alpha^2)} \left(\frac{a}{r_{ib}}\right)^2} \quad (505.6-2)$$

where

$\left(\frac{KL}{r}\right)_m$ = modified column slenderness of built-up member

$\left(\frac{KL}{r}\right)_o$ = column slenderness of built-up member acting as a unit in the buckling direction being considered

a = distance between connectors, mm

r_i = minimum radius of gyration of individual component, mm

r_{ib} = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, mm

α = separation ratio = $h/2r_{ib}$

h = distance between centroids of individual components perpendicular to the member axis of buckling, mm

2. The nominal compressive strength of built-up members composed of two or more shapes or plates with at least one open side interconnected by perforated cover plates or lacing with tie plates shall be determined in accordance with Sections 505.3, 505.4, or 505.7 subject to the modification given in Section 505.6.1(1).

505.6.2 Dimensional Requirements

Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, a , such that the effective slenderness ratio K_a/r_i of each of the component shapes, between the fasteners, does not exceed three-fourths times the governing slenderness ratio of the built-up member. The least radius of gyration, r_i , shall be used in computing the slenderness ratio of each component part. The end connection shall be welded or pretensioned bolted with Class A or B faying surfaces.

User Note: It is acceptable to design a bolted end connection of a built-up compression member for the full compressive load with bolts in shear and bolt values based on bearing values; however, the bolts must be pretensioned. The requirement for Class A or B faying surfaces is not intended for the resistance of the axial force in the built-up member,

but rather to prevent relative movement between the components at the end as the built-up member takes a curved shape.

At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1 1/2 times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds or bolts shall be adequate to provide for the transfer of the required forces. For limitations on the longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates, see Section 510.3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the

thinner outside plate times $0.75 \sqrt{E/F_y}$, nor 300 mm,

when intermittent welds are provided along the edges of the components or when fasteners are provided on all gage lines at each section. When fasteners are staggered, the maximum spacing on each gage line shall not exceed the

thickness of the thinner outside plate times $1.12 \sqrt{E/F_y}$ nor 450 mm.

Open sides of compression members built up from plates or shapes shall be provided with continuous cover plates perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Section 502.4, is assumed to contribute to the available strength provided the following requirements are met:

1. The width-thickness ratio shall conform to the limitations of Section 502.4.

User Note: It is conservative to use the limiting width/thickness ratio for Case 14 in Table 502.4.2 with the width, b , taken as the transverse distance between the nearest lines of fasteners. The net area of the plate is taken at the widest hole. In lieu of this approach, the limiting width thickness ratio may be determined through analysis

2. The ratio of length (in direction of stress) to width of hole shall not exceed two.

3. The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.
4. The periphery of the holes at all points shall have a minimum radius of 38 mm.

As an alternative to perforated cover plates, lacing with tie plates is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In members providing available strength, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall total not less than one-third the length of the plate. In bolted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, shall be so spaced that the L/r ratio of the flange included between their connections shall not exceed three-fourths times the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to 2 percent of the available compressive strength of the member. The L/r ratio for lacing bars arranged in single systems shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. For lacing bars in compression, l is permitted to be taken as the unsupported length of the lacing bar between welds or fasteners connecting it to the components of the built-up member for single lacing, and 70 percent of that distance for double lacing.

User Note: The inclination of lacing bars to the axis of the member shall preferably be not less than 60° for single lacing and 45° for double lacing. When the distance between the lines of welds or fasteners in the flanges is more than 380 mm, the lacing shall preferably be double or be made of angles.

For additional spacing requirements, see Section 510.3.5.

505.7 Members with Slender Elements

This section applies to compression members with slender sections, as defined in Section 502.4 for uniformly compressed elements.

The nominal compressive strength, P_n , shall be determined based on the limit states of flexural, torsional and flexural-torsional buckling.

$$P_n = F_{cr} A_g \quad (505.7-1)$$

$$\text{a. When } \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{QF_y}} \quad (\text{or } F_e \geq 0.44QF_y)$$

$$F_{cr} = Q \left[0.658 \frac{QF_y}{F_e} \right] F_y \quad (505.7-2)$$

$$\text{b. When } \frac{KL}{r} > 4.71 \sqrt{\frac{E}{QF_y}} \quad (\text{or } F_e < 0.44QF_y)$$

$$F_{cr} = 0.877F_e \quad (505.7-3)$$

F_e = elastic critical buckling stress, calculated using Eqs. 505.3-4 and 505.4-4 for doubly symmetric members, Eqs. 505.3-4 and 505.4-5 for singly symmetric members, and Eq. 505.4-6 for unsymmetric members, except for single angles where F_e is calculated using Eq. 505.3-4.

Q = 1.0 for members with compact and noncompact sections, as defined in Section 502.4, for uniformly compressed elements

= $Q_s Q_a$ for members with slender-element sections, as defined in Section 502.4, for uniformly compressed elements.

User Note: For cross sections composed of only unstiffened slender elements, $Q_a = 1$ then $Q_s = Q$. For cross sections composed of only stiffened slender elements, $Q_s = 1$ then $Q = Q_a$. For cross sections composed of both stiffened and unstiffened slender elements, $Q = Q_s Q_a$.

505.7.1 Slender Unstiffened Elements, Q_s

The reduction factor Q_s for slender unstiffened elements is defined as follows:

1. For flanges, angles, and plates projecting from rolled columns or other compression members:

$$\text{a. When } \frac{b}{t} \leq 0.56 \sqrt{\frac{E}{F_y}}$$

$$Q_s = 1.0 \quad (505.7-4)$$

b. When $0.56 \sqrt{\frac{E}{F_y}} < \frac{b}{t} < 1.03 \sqrt{\frac{E}{F_y}}$

$$Q_s = 1.415 - 0.74 \left(\frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \quad (505.7-5)$$

c. When $\frac{b}{t} \geq 1.03 \sqrt{\frac{E}{F_y}}$

$$Q_s = \frac{0.69E}{F_y \left(\frac{b}{t} \right)^2} \quad (505.7-6)$$

2. For flanges, angles, and plates projecting from built-up columns or other compression members:

a. When $\frac{b}{t} \leq 0.64 \sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = 1.0 \quad (505.7-7)$$

b. When $0.64 \sqrt{\frac{Ek_c}{F_y}} < \frac{b}{t} \leq 1.17 \sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = 1.415 - 0.65 \left(\frac{b}{t} \right) \sqrt{\frac{F_y}{Ek_c}} \quad (505.7-8)$$

c. When $\frac{b}{t} > 1.17 \sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = \frac{0.90Ek_c}{F_y \left(\frac{b}{t} \right)^2} \quad (505.7-9)$$

where:

$$k_c = \frac{4}{\sqrt{h/t_w}} \quad \text{and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes}$$

3. For single angles

a. When $\frac{b}{t} \leq 0.45 \sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \quad (505.7-10)$$

b. When $0.45 \sqrt{\frac{E}{F_y}} < \frac{b}{t} \leq 0.91 \sqrt{\frac{E}{F_y}}$

$$Q_s = 1.34 - 0.76 \left(\frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \quad (505.7-11)$$

c. When $\frac{b}{t} > 0.91 \sqrt{\frac{E}{F_y}}$

$$Q_s = \frac{0.53E}{F_y \left(\frac{b}{t} \right)^2} \quad (505.7-12)$$

where

b = full width of longest angle leg, mm

4. For stems of tees

a. When $\frac{d}{t} \leq 0.75 \sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \quad (505.7-13)$$

b. When $0.75 \sqrt{\frac{E}{F_y}} < \frac{d}{t} \leq 1.03 \sqrt{\frac{E}{F_y}}$

$$Q_s = 1.908 - 1.22 \left(\frac{d}{t} \right) \sqrt{\frac{F_y}{E}} \quad (505.7-14)$$

c. When $\frac{d}{t} > 1.03 \sqrt{\frac{E}{F_y}}$

$$Q_s = \frac{0.69E}{F_y \left(\frac{d}{t} \right)^2} \quad (505.7-15)$$

where:

b = width of unstiffened compression element, as defined in Section 502.4, mm

d = the full nominal depth of tee, mm

t = thickness of element, mm

505.7.2. Slender Stiffened Elements, Q_a

The reduction factor, Q_a for slender stiffened elements is defined as follows:

$$Q_a = \frac{A_{eff}}{A} \quad (505.7-16)$$

where:

A = total cross-sectional area of member, mm²
 A_{eff} = summation of the effective areas of the cross section based on the reduced effective width, b_e , mm²

The reduced effective width, b_e , is determined as follows:

1. For uniformly compressed slender elements, with

$$\frac{b}{t} \geq 1.49 \sqrt{\frac{E}{f}}, \text{ except flanges of square and rectangular sections of uniform thickness:}$$

$$b_e = 1.92t \sqrt{\frac{E}{f}} \left[1 - \frac{0.34}{(b/t)} \sqrt{\frac{E}{f}} \right] \leq b \quad (505.7-17)$$

where

f is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$.

2. For flanges of square and rectangular slender-element sections of uniform thickness with

$$\frac{b}{t} \geq 1.40 \sqrt{\frac{E}{f}}:$$

$$b_e = 1.92t \sqrt{\frac{E}{f}} \left[1 - \frac{0.38}{(b/t)} \sqrt{\frac{E}{f}} \right] \leq b \quad (505.7-18)$$

where:

$$f = P_n / A_{eff}$$

User Note: In lieu of calculating $f = P_n / A_{eff}$ which requires iteration, f may be taken equal to F_y . This will result in a slightly conservative estimate of column capacity.

1. For axially-loaded circular sections:

$$\text{When } 0.11 \frac{E}{F_y} < \frac{D}{t} < 0.45 \frac{E}{F_y}$$

$$Q = Q_a = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \quad (505.7-19)$$

where

D = outside diameter, mm
 t = wall thickness, mm

SECTION 506

DESIGN OF MEMBERS FOR FLEXURE

506.1 General Provisions

The design flexural strength, $\phi_b M_n$, and the allowable flexural strength, M_n/Ω_b , shall be determined as follows:

1. For all provisions in this section

$$\phi_b = 0.90(LRFD) \quad \Omega_b = 1.67(ASD)$$

and the nominal flexural strength, M_n , shall be determined according to Section 506.2 through Section 506.12.

2. The provisions in this section are based on the assumption that points of support for beams and girders are restrained against rotation about their longitudinal axis.

The following terms are common to the equations in this section except where noted:

C_b = lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the unsupported segment are braced

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} R_m \leq 3.0$$

(506.1-1)

where:

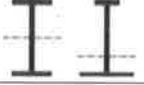
M_{max}	=	absolute value of maximum moment in the unbraced segment, N-mm
M_A	=	absolute value of moment at quarter point of the unbraced segment, N-mm
M_B	=	absolute value of moment at centerline of the unbraced segment, N-mm
M_C	=	absolute value of moment at three-quarter point of the unbraced segment, N-mm
R_m	=	cross-section monosymmetry parameter
	=	1.0, doubly symmetric members
	=	1.0, singly symmetric members subjected to single curvature bending

In singly symmetric members subjected to reverse curvature bending, the lateral-torsional buckling strength shall be checked for both flanges. The available flexural strength shall be greater than or equal to the maximum required moment causing compression within the flange under consideration.

C_b is permitted to be conservatively taken as 1.0 for all cases. For cantilevers or overhangs where the free end is unbraced, $C_b = 1.0$.

User Note: For doubly symmetric members with no transverse loading between brace points, Eq. 506.1-1 reduces to 2.27 for the case of equal end moments of opposite sign and to 1.67 when one end moment equals zero.

Table 506.1.1 Selection Table for the Application of Section 506 Sections

Section in Section 506	Cross Section	Flange Slenderness	Web Slenderness	Limit States
506.2		C	C	Y, LTB
506.3		NC, S	C	LTB, FLB
506.4		C, NC, S	C, NC	Y, LTB, FLB, TFY
506.5		C, NC, S	S	Y, LTB, FLB, TFY
506.6		C, NC, S	N/A	Y, FLB
506.7		C, NC, S	C, NC	Y, FLB, WLB
506.8		N/A	N/A	Y, LB
506.9		C, NC, S	N/A	Y, LTB, FLB
506.10		N/A	N/A	Y, LTB, LLB
506.11		N/A	N/A	Y, LTB
506.12	Unsymmetrical shapes	N/A	N/A	All limit states

Y = yielding, LTB = lateral-torsional buckling, FLB = flange local buckling, WLB = web local buckling, TFY = tension flange yielding, LLB = leg local buckling, LB = local buckling, C = compact, NC = noncompact, S = slender

506.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent about their Major Axis

This section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Section 502.4.

The nominal flexural strength, M_n , shall be the lower value obtained according to the limit states of yielding (plastic moment) and lateral-torsional buckling.

506.2.1. Yielding

$$M_n = M_p = F_y Z_x \quad (506.2-1)$$

where

F_y = specified minimum yield stress of the type of steel being used, MPa

Z_x = plastic section modulus about the x-axis, mm³

506.2.2 Lateral-Torsional Buckling

1. When $L_b \leq L_p$ the limit state of lateral-torsional buckling does not apply.

2. When $L_p < L_b \leq L_r$

$$M_n = C_b \left[M_p - (M_p - 0.7 F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad (506.2-2)$$

3. When $L_b > L_r$

$$M_n = F_{cr} S_x \leq M_p \quad (506.2-3)$$

where

L_b = Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}} \right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}} \right)^2} \quad (506.2-4)$$

E = modulus of elasticity of steel = 200,000 MPa

Jc = torsional constant, mm⁴

S_x = elastic section modulus taken about the x-axis, mm³

User Note: The square root term in Eq. 506.2-4 may be conservatively taken equal to 1.0.

The limiting lengths L_p and L_r are determined as follows:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \quad (506.2-5)$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7 F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7 F_y S_x h_o}{E Jc} \right)^2}} \quad (506.2-6)$$

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \quad (506.2-7)$$

and

For a doubly symmetric I-shape: $c = 1$ (506.2-8a)

For a channel: $c = \frac{h_o}{2} \sqrt{\frac{I_y}{C_w}}$ (506.2-8b)

where

h_o = distance between the flange centroids, mm

User Note: If the square root term in Eq. 506.2-4 is conservatively taken equal to 1, Eq. 506.2-6 becomes

$$L_r = \pi r_{ts} \sqrt{\frac{E}{0.7 F_y}}$$

Note that this approximation can be extremely conservative.

For doubly symmetric I-shapes with rectangular flanges,

$C_w = \frac{I_y h_o^2}{4}$ and thus Eq. 506.2-7 becomes

$$r_{ts}^2 = \frac{I_y h_o}{2 S_x}$$

r_{ts} may be approximated accurately and conservatively as the radius of gyration of the compression flanges plus one-sixth of the web:

$$r_{ts} = \sqrt{\frac{b_f}{12 \left(1 + \frac{1}{6} \frac{h t_w}{b_f t_f} \right)}}$$

506.3 Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about their Major Axis

This section applies to doubly symmetric I-shaped members bent about their major axis having compact webs and noncompact or slender flanges as defined in Section 502.4.

The nominal flexural strength, M_n , shall be the lower value obtained according to the limit states of lateral-torsional buckling and compression flange local buckling.

506.3.1 Lateral-Torsional Buckling

For lateral-torsional buckling, the provisions of Section 506.2.2 shall apply.

506.3.2 Compression Flange Local Buckling

1. For sections with noncompact flanges

$$M_n = \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (506.3-1)$$

2. For sections with slender flanges

$$M_n = \frac{0.9Ek_c S_x}{\lambda^2} \quad (506.3-2)$$

where

$$\lambda = \frac{b_f}{2t_f}$$

$\lambda_{pf} = \lambda_p$ is the limiting slenderness for a compact flange, Table 502.4.1 and Table 502.4.2

$\lambda_{rf} = \lambda_r$ is the limiting slenderness for a noncompact flange, Table 502.4.1 and Table 502.4.2

$$k_c = \frac{4}{\sqrt{h/t_w}} \quad \text{and shall not be taken less than}$$

0.35 nor greater than 0.76 for calculation purposes.

506.4 Other I-Shaped Members with Compact or Noncompact Webs Bent about their Major Axis

This section applies to: (a) doubly symmetric I-shaped members bent about their major axis with noncompact webs; and (b) singly symmetric I-shaped members with webs attached to the mid-width of the flanges, bent about their major axis, with compact or noncompact webs, as defined in Section 502.4.

User Note: I-shaped members for which this section is applicable may be designed conservatively using Section 506.5.

The nominal flexural strength, M_n , shall be the lowest value obtained according to the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling and tension flange yielding.

506.4.1 Compression Flange Yielding

$$M_n = R_{pc} M_{yc} = R_{pc} F_y S_{xc} \quad (506.4-1)$$

506.4.2 Lateral-Torsional Buckling

1. When $L_b < L_p$ the limit state of lateral-torsional buckling does not apply.

2. When $L_p < L_b \leq L_r$

$$M_n = C_b \left[R_{pc} M_{yc} - (R_{pc} M_{yc} - F_L S_{xc}) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq R_{pc} M_{yc} \quad (506.4-2)$$

3. When $L_b > L_r$

$$M_n = F_{cr} S_{xc} \leq R_{pc} M_{yc} \quad (506.4-3)$$

where

$$M_{yc} = F_y S_{xc} \quad (506.4-4)$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \left(\frac{L_b}{r_t} \right)^2} \quad (506.4-5)$$

For $\frac{I_{yc}}{I_y} \leq 0.23$, J shall be taken as zero.

The stress, F_L , is determined as follows:

$$\text{For } \frac{S_{xt}}{S_{xc}} \geq 0.7$$

$$F_L = 0.7F_y \quad (506.4-6a)$$

$$\text{For } \frac{S_{xt}}{S_{xc}} < 0.7$$

$$F_L = F_y \frac{S_{xt}}{S_{xc}} \geq 0.5F_y \quad (506.4-6b)$$

The limiting laterally unbraced length for the limit state of yielding, L_p , is

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \quad (506.4-7)$$

The limiting unbraced length for the limit state of inelastic lateral-torsional buckling, L_r , is

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc} h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{F_L S_{xc} h_o}{E J} \right)^2}} \quad (506.4-8)$$

The web plastification factor, R_{pc} , is determined as follows:

a. For $\frac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pc} = \frac{M_p}{M_{yc}} \quad (506.4-9a)$$

b. For $\frac{h_c}{t_w} > \lambda_{pw}$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1 \right) \left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yc}} \quad (506.4-9b)$$

where

$$M_p = Z_x F_y \leq 1.6 S_{xc} F_y$$

S_{xc}, S_{xt} = elastic section modulus referred to tension and compression flanges, respectively, mm³

$$\lambda = \frac{h_c}{t_w}$$

λ_{pw} = λ_p , the limiting slenderness for a compact web, see Tables 502.4.1 and 502.4.2

λ_{rw} = λ_r , the limiting slenderness for a noncompact web, see Tables 502.4.1 and 502.4.2

The effective radius of gyration for lateral-torsional buckling, r_t , is determined as follows:

1. For I-shapes with a rectangular compression flange:

$$r_t = \sqrt{\frac{b_{fc}}{12 \left(\frac{h_o}{d} + \frac{1}{6} a_w \frac{h^2}{h_o d} \right)}} \quad (506.4-10)$$

where

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \quad (506.4-11)$$

b_{fc} = compression flange width, mm

t_{fc} = compression flange thickness, mm

2. For I-shapes with channel caps or cover plates attached to the compression flange:

r_t = radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, mm

a_w = the ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components

User Note: For I-shapes with a rectangular compression flange, r_t may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-third of the compression portion of the web; in other words,

$$r_t = \frac{b_{fc}}{\sqrt{12 \left(1 + \frac{1}{6} a_w \right)}}$$

506.4.3 Compression Flange Local Buckling

1. For sections with compact flanges, the limit state of local buckling does not apply.

2. For sections with noncompact flanges

$$M_n = \left[R_{pc} M_{yc} - (R_{pc} M_{yc} - F_L S_{xc}) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (506.4-12)$$

3. For sections with slender flanges

$$M_n = \frac{0.9 E k_c S_{xc}}{\lambda^2} \quad (506.4-13)$$

where:

F_L = defined in Eqs. 506.4-6a and 506.4-6b

R_{pc} = the web plastification factor, determined by Eqs. 506.4-9

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

- $\lambda = (b_{fc}/2t_{fc})$
 $\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Tables 502.4.1 and 502.4.2
 $\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Tables 502.4.1 and 502.4.2

506.4.4 Tension Flange Yielding

1. When $S_{xt} \geq S_{xc}$ the limit state of tension flange yielding does not apply.
2. When $S_{xt} < S_{xc}$

$$M_n = R_{pt}M_{yt} \quad (506.4-14)$$

where

$$M_{yt} = F_{yt}S_{xt}$$

The web plastification factor corresponding to the tension flange yielding limit state, R_{pt} is determined as follows:

- a. For $\frac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pt} = \frac{M_p}{M_{yt}} \quad (506.4-15a)$$

- b. For $\frac{h_c}{t_w} > \lambda_{pw}$

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1 \right) \left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yt}} \quad (506.4-15b)$$

where

- $\lambda = h_c/t_w$
 $\lambda_{pw} = \lambda_p$, the limiting slenderness for a compact web, defined in Tables 502.4.1 and 502.4.2
 $\lambda_{rw} = \lambda_r$, the limiting slenderness for a non compact web, defined in Tables 502.4.1 and 502.4.2

506.5 Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about their Major Axis

This section applies to doubly symmetric and singly symmetric I-shaped members with slender webs attached to the mid-width of the flanges, bent about their major axis, as defined in Section 502.4

The nominal flexural strength, M_n , shall be the lowest value obtained according to the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling and tension flange yielding.

506.5.1 Compression Flange Yielding

$$M_n = R_{pg}F_yS_{xc} \quad (506.5-1)$$

506.5.2 Lateral-Torsional Buckling

$$M_n = R_{pg}F_{cr}S_{xc} \quad (506.5-2)$$

1. When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.
2. When $L_p < L_b \leq L_r$

$$F_{cr} = C_b \left[F_y - (0.3F_y) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq F_y \quad (506.5-3)$$

3. When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \leq F_y \quad (506.5-4)$$

where

L_p is defined by Eq. 506.4-7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} \quad (506.5-5)$$

R_{pg} is the bending strength reduction factor:

$$R_{pg} = 1 - \frac{a_w}{1200 + 300a_w} \left(\frac{h_c}{t_w} - 5.7 \sqrt{\frac{E}{F_y}} \right) \leq 1.0 \quad (506.5-6)$$

where:

a_w = defined by Eq. 506.4-11 but shall not exceed 10 and
 r_t = the effective radius of gyration for lateral buckling
 as defined in Section 506.4.

506.5.3 Compression Flange Local Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \quad (506.5-7)$$

1. For sections with compact flanges, the limit state of compression flange local buckling does not apply.

2. For sections with noncompact flanges

$$F_{ct} = \left[F_y - (0.3 F_y) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (506.5-8)$$

3. For sections with slender flange sections

$$F_{cr} = \frac{0.9 E k_c}{\left(\frac{b_f}{2 t_f} \right)^2} \quad (506.5-9)$$

where

$$k_c = \frac{4}{\sqrt{h/t_w}} = \text{and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes}$$

$$\lambda = b_{fc} / 2 t_{fc}$$

$$\lambda_{pf} = \lambda_p, \text{ the limiting slenderness for a compact flange, Tables 502.4.1 and 502.4.2}$$

$$\lambda_{rf} = \lambda_r, \text{ the limiting slenderness for a noncompact flange, Tables 502.4.1 and 502.4.2}$$

506.5.4 Tension Flange Yielding

1. When $S_{xt} \geq S_{xc}$, the limit state of tension flange yielding does not apply.

2. When $S_{xt} < S_{xc}$

$$M_n = F_y S_{xt} \quad (506.5-10)$$

506.6 I-Shaped Members and Channels Bent about their Minor Axis

This section applies to I-shaped members and channels bent about their minor axis.

The nominal flexural strength, M_n , shall be the lower value obtained according to the limit states of yielding (plastic moment) and flange local buckling.

506.6.1 Yielding

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \quad (506.6-1)$$

506.6.2 Flange Local Buckling

1. For Sections with compact flanges the limit state of yielding shall apply.

User Note: All current ASTM A6 W, S, M, C and MC shapes except W21x48, W14x99, W14x90, W12x65, W10x12, W8x31, W8x10, W6x15, W6x9, W6x8.5, and M4x6 have compact flanges at $F_y = 345$ Mpa.

2. For sections with noncompact flanges

$$M_n = \left[M_p - (M_p - 0.7 F_y S_y) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (506.6-2)$$

3. For sections with slender flanges

$$M_n = F_{cr} S_y \quad (506.6-3)$$

where

$$F_{cr} = \frac{0.69 E}{\left(\frac{b_f}{2 t_f} \right)^2} \quad (506.6-4)$$

$\lambda = b/t$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Tables 502.4.1 and 502.4.2

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Tables 502.4.1 and 502.4.2

S_y = for a channel shall be taken as the minimum section modulus

506.7 Square and Rectangular HSS and Box-shaped Members

This section applies to square and rectangular HSS, and doubly symmetric box-shaped members bent about either axis, having compact or noncompact webs and compact, noncompact or slender flanges as defined in Section 502.4.

The nominal flexural strength, M_n , shall be the lowest value obtained according to the limit states of yielding (plastic moment), flange local buckling and web local buckling under pure flexure.

506.7.1 Yielding

$$M_n = M_p = F_y Z \quad (506.7-1)$$

where:

Z = plastic section modulus about the axis of bending, mm^3

506.7.2 Flange Local Buckling

1. For compact sections, the limit state of flange local buckling does not apply.
2. For sections with noncompact flanges

$$M_n = M_p - (M_p - F_y S) \left(3.57 \frac{b}{t} \sqrt{\frac{F_y}{E}} - 4.0 \right) \leq M_p \quad (506.7-2)$$

3. For sections with slender flanges

$$M_n = F_y S_{eff} \quad (506.7-3)$$

where

S_{eff} is the effective section modulus determined with the effective width of the compression flange taken as:

$$b_e = 1.92t \sqrt{\frac{E}{F_y}} \left[1 - \frac{0.38}{b/t} \sqrt{\frac{E}{F_y}} \right] \leq b \quad (506.7-4)$$

506.7.3 Web Local Buckling

1. For compact sections, the limit state of web local buckling does not apply.
2. For sections with noncompact webs

$$M_n = M_p - (M_p - F_y S_x) \left(0.305 \frac{h}{t_w} \sqrt{\frac{F_y}{E}} - 0.738 \right) \leq M_p \quad (506.7-5)$$

506.8 Round HSS

This section applies to round HSS having D/t ratios of less than $\frac{0.45E}{F_y}$.

The nominal flexural strength, M_n , shall be the lower value obtained according to the limit states of yielding (plastic

moment) and local buckling.

506.8.1 Yielding

$$M_n = M_p = F_y Z \quad (506.8-1)$$

506.8.2 Local Buckling

1. For compact sections, the limit state of flange local buckling does not apply.
2. For noncompact sections

$$M_n = \left(\frac{0.021E}{\frac{D}{t}} + F_y \right) S \quad (506.8-2)$$

3. For sections with slender walls

$$M_n = F_{cr} S \quad (506.8-3)$$

where

$$F_{cr} = \frac{0.33E}{\frac{D}{t}} \quad (506.8-4)$$

S = elastic section modulus, mm^3

506.9 Tees and Double Angles Loaded in the Plane of Symmetry

This section applies to tees and double angles loaded in the plane of symmetry.

The nominal flexural strength, M_n , shall be the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling and flange local buckling.

506.9.1 Yielding

$$M_n = M_p \quad (506.9-1)$$

where

$$M_p = F_y Z_x \leq 1.6 M_y \quad \text{for stems in tension} \quad (506.9-2)$$

$$\leq M_y \quad \text{for stems in compression} \quad (506.9-3)$$

506.9.2 Lateral-Torsional Buckling

$$M_n = M_{cr} = \frac{\pi \sqrt{EI_y GJ}}{L_b} \left[B + \sqrt{1 + B^2} \right] \quad (506.9-4)$$

where:

$$B = \pm 2.3 \left(\frac{d}{L_b} \right) \sqrt{\frac{I_y}{J}} \quad (506.9-5)$$

The plus sign for B applies when the stem is in tension and the minus sign applies when the stem is in compression. If the tip of the stem is in compression anywhere along the unbraced length, the negative value of B shall be used.

506.9.3 Flange Local Buckling of Tees

$$M_n = F_{cr} S_{xc} \quad (506.9-6)$$

S_{xc} is the elastic section modulus referred to the compression flange.

F_{cr} is determined as follows:

1. For compact sections, the limit state of flange local buckling does not apply.
2. For noncompact sections

$$F_{cr} = F_y \left[1.19 - 0.50 \left(\frac{b_f}{2t_f} \right) \sqrt{\frac{F_y}{E}} \right] \quad (506.9-7)$$

3. For slender sections

$$F_{cr} = \frac{0.69E}{\left(\frac{b_f}{2t_f} \right)^2} \quad (506.9-8)$$

506.10 Single Angles

This section applies to single angles with and without continuous lateral restraint along their length.

Single angles with continuous lateral-torsional restraint along the length shall be permitted to be designed on the basis of geometric axis (x, y) bending. Single angles without continuous lateral-torsional restraint along the length shall be designed using the provisions for principal axis bending except where the provision for bending about a geometric axis is permitted.

User Note: For geometric axis design, use section properties computed about the x - and y -axis of the angle, parallel and perpendicular to the legs. For principal axis design use section properties computed about the major and minor principal axes of the angle.

The nominal flexural strength, M_n , shall be the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling and leg local buckling.

506.10.1 Yielding

$$M_n = 1.5M_y \quad (506.10-1)$$

where:

M_y = yield moment about the axis of bending, N-mm.

506.10.2 Lateral-Torsional Buckling

For single angles without continuous lateral-torsional restraint along the length

1. When $M_e \leq M_y$

$$M_n = \left(0.92 - \frac{0.17M_e}{M_y} \right) M_e \quad (506.10-2)$$

2. When $M_e > M_y$

$$M_n = \left(1.92 - 1.17 \sqrt{\frac{M_y}{M_e}} \right) M_y \leq 1.5M_y \quad (506.10-3)$$

where:

M_e , the elastic lateral-torsional buckling moment, is determined as follows:

1. For bending about one of the geometric axes of an equal-leg angle with no lateral-torsional restraint.

- a. With maximum compression at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L^2} \left(\sqrt{1 + 0.78 \left(\frac{Lt}{b^2} \right)^2} - 1 \right) \quad (506.10-4a)$$

- b. With maximum tension at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L^2} \left(\sqrt{1 + 0.78 \left(\frac{Lt}{b^2} \right)^2} + 1 \right) \quad (506.10-4b)$$

M_y shall be taken as 0.80 times the yield moment calculated using the geometric section modulus.

User Note: M_n may be taken as M_y for single angles with their vertical leg toe in compression, and having a span-to-depth ratio less than or equal to

$$\frac{1.64E}{F_y} \sqrt{\left(\frac{t}{b}\right)^2 - 1.4 \frac{F_y}{E}}$$

2. For bending about one of the geometric axes of an equal-leg angle with lateral-torsional restraint at the point of maximum moment only

M_e shall be taken as 1.25 times M_e computed using Eq. 506.10-4a or 506.10-4b

M_y shall be taken as the yield moment calculated using the geometric section modulus.

3. For bending about the major principal axis of equal-leg angles:

$$M_e = \frac{0.46Eb^2t^2C_b}{L} \quad (506.10-5)$$

4. For bending about the major principal axis of unequal-leg angles:

$$M_e = \frac{4.9EI_zC_b}{L^2} \left(\sqrt{\beta_w^2 + 0.052 \left(\frac{Lt}{r_z}\right)^2} + \beta_w \right) \quad (506.10-6)$$

where:

C_b is computed using Eq. 506.1-1 with a maximum value of 1.5.

L = laterally unbraced length of a member, mm.

I_z = minor principal axis moment of inertia, mm⁴.

r_z = radius of gyration for the minor principal axis, mm

t = angle leg thickness, mm

β_w = a section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of β_w shall be used

506.10.3 Leg Local Buckling

The limit state of leg local buckling applies when the toe of the leg is in compression.

1. For compact sections, the limit state of leg local buckling does not apply.
2. For sections with noncompact legs

$$M_n = F_y S_c \left[2.43 - 1.72 \left(\frac{b}{t}\right) \sqrt{\frac{F_y}{E}} \right] \quad (506.10-7)$$

3. For sections with slender legs

$$M_n = F_{cr} S_c \quad (506.10-8)$$

where:

$$F_{cr} = \frac{0.71E}{\left(\frac{b}{t}\right)^2}$$

b = outside width of leg in compression, mm

S_c = elastic section modulus to the toe in compression relative to the axis of bending, mm³. For bending about one of the geometric axes of an equal-leg angle with no lateral-torsional restraint, S_c shall be 0.80 of the geometric axis section modulus.

506.11 Rectangular Bars and Rounds

This section applies to rectangular bars bent about either geometric axis and rounds.

The nominal flexural strength, M_n , shall be the lower value obtained according to the limit states of yielding (plastic moment) and lateral-torsional buckling, as required.

506.11.1 Yielding

For rectangular bars with $\frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}$ bent about their major axis, rectangular bars bent about their minor axis, and rounds:

$$M_n = M_p = F_y Z \leq 1.6 M_y \quad (506.11-1)$$

506.11.2 Lateral-Torsional Buckling

1. For rectangular bars with $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \leq \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \frac{F_y}{E} \right] M_y \leq M_p \quad (506.11-2)$$

2. For rectangular bars with $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = F_{cr} S_x \leq M_p \quad (506.11-3)$$

where:

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \quad (506.11-4)$$

- t = width of rectangular bar parallel to axis of bending, mm
 d = depth of rectangular bar, in. mm
length between points that are either braced against
 L_b = lateral displacement of the compression region or braced against twist of the cross section, mm
3. For round and rectangular bars bent about their minor axis, the limit state of lateral-torsional buckling need not be considered.

506.12 Unsymmetrical Shapes

This section applies to all unsymmetrical shapes, except single angles.

The nominal flexural strength, M_n , shall be the lowest value obtained according to the limit state of yielding (yield moment), lateral-torsional buckling and local buckling where

$$M_n = F_n S \quad (506.12-1)$$

where:

S = lowest elastic section modulus relative to the axis of bending, mm³

506.12.1 Yielding

$$F_n = F_y \quad (506.12-2)$$

506.12.2 Lateral-Torsional Buckling

$$F_n = F_{cr} \leq F_y \quad (506.12-3)$$

where:

F_{cr} = buckling stress for the section as determined by analysis, MPa

User Note: In the case of z-shaped members, it is recommended that F_{cr} be taken as **0.50 F_{cr}** of a channel with same flange and web properties.

506.12.3 Local Buckling

$$F_n = F_{cr} \leq F_y \quad (506.12-4)$$

where:

F_{cr} = buckling stress for the section as determined by analysis, MPa

506.13 Proportions of Beams and Girders**506.13.1 Hole Reductions**

This section applies to rolled or built-up shapes, and cover-plated beams with holes, proportioned on the basis of flexural strength of the gross section.

In addition to the limit states specified in the previous sections, the nominal flexural strength, M_n , shall be limited according to the limit state of tensile rupture of the tension flange.

- For $F_u A_{fn} \geq Y_t F_y A_{fg}$, the limit state of tensile Rupture shall not apply.
- For $F_u A_{fn} < Y_t F_y A_{fg}$, the nominal flexural strength, M_n , at the location of the holes in the tension flange shall not be taken greater than:

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \quad (506.13-1)$$

where:

A_{fg} = gross tension flange area, shall be calculated in accordance with the provisions of Section 504.3.1, mm²

A_{fn} = net tension flange area, calculated in accordance with the provisions of Section 504.3.2, mm²

Y_t = 1.0 for $F_y/F_u \leq 0.80$
= 1.1 otherwise

506.13.2 Proportioning Limits for I-Shaped Members

Singly symmetric I-shaped members shall satisfy the following limit:

$$0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9 \quad (506.13-2)$$

I-shaped members with slender webs shall also satisfy the following limits:

1. For $\frac{a}{h} \leq 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = 11.7 \sqrt{\frac{E}{F_y}} \quad (506.13-3)$$

2. For $\frac{a}{h} > 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = \frac{0.42E}{F_y} \quad (506.13-4)$$

where:

a = clear distance between transverse stiffeners, mm

In unstiffened girders h/t_w shall not exceed 260. The ratio of the web area to the compression flange area shall not exceed 10.

506.13.3 Cover Plates

Flanges of welded beams or girders may be varied in thickness or width by splicing a series of plates or by the use of cover plates.

The total cross-sectional area of cover plates of bolted girders shall not exceed 70 percent of the total flange area.

High-strength bolts or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total horizontal shear resulting from the bending forces on the girder. The longitudinal distribution of these bolts or intermittent welds shall be in proportion to the intensity of the shear. However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Section 505.6 or 504.4, respectively. Bolts or welds connecting flange to web shall also be proportioned to transmit to the web any loads applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial-length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical connection or fillet welds. The attachment shall be adequate, at the applicable strength given in Sections 510.2.2, 510.3.8, or 502.3.9 to develop the cover plate's portion of the flexural strength in the beam or girder at the theoretical cutoff point.

For welded cover plates, the welds connecting the cover plate termination to the beam or girder shall have continuous welds along both edges of the cover plate in the length a' , defined below, and shall be adequate to develop the cover plate's portion of the strength of the beam or girder at the distance a' from the end of the cover plate.

1. When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate

$$a' = w \quad (506.13-5)$$

where:

w = width of cover plate, mm

2. When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate

$$a' = 1.5w \quad (506.13-6)$$

3. When there is no weld across the end of the plate

$$a' = 2w \quad (506.13-7)$$

506.13.4. Built-Up Beams

Where two or more beams or channels are used side-by-side to form a flexural member, they shall be connected together in compliance with Section 505.6.2. When concentrated loads are carried from one beam to another, or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be welded or bolted between the beams.

SECTION 507

DESIGN OF MEMBERS FOR SHEAR

507.1 General Provisions

Two methods of calculating shear strength are presented below. The method presented in Section 507.2 does not utilize the post buckling strength of the member (tension field action). The method presented in Section 507.3 utilizes tension field action.

The design shear strength, $\phi_v V_n$, and the allowable shear strength, V_n/Ω_v , shall be determined as follows.

For all provisions in this section except Section 507.2.1(1):

$$\phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

507.2 Members with Unstiffened or Stiffened Webs

507.2.1 Nominal Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The nominal shear strength, V_n , of unstiffened or stiffened webs, according to the limit states of shear yielding and shear buckling, is

$$V_n = 0.6F_y A_w C_v \quad (507.2-1)$$

1. For webs of rolled I-shaped members

$$\text{with } h/t_w \leq 2.24 \sqrt{E/F_y}$$

$$\phi_v = 1.00 \text{ (LRFD)} \quad \Omega_v = 1.50 \text{ (ASD)}$$

and

$$C_v = 1.0 \quad (507.2-2)$$

2. For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round HSS, the web shear coefficient, C_v , is determined as follows:

$$\text{a. For } h/t_w \leq 1.10 \sqrt{k_v E/F_y}:$$

$$C_v = 1.0 \quad (507.2-3)$$

$$\text{b. For } 1.10 \sqrt{k_v E/F_y} < h/t_w \leq 1.37 \sqrt{k_v E/F_y}$$

$$C_v = \frac{1.10 \sqrt{k_v E/F_y}}{h/t_w} \quad (507.2-4)$$

$$\text{c. For } h/t_w > 1.37 \sqrt{k_v E/F_y}$$

$$C_v = \frac{1.51 E k_v}{(h/t_w)^2 F_y} \quad (507.2-5)$$

where:

A_w = the overall depth times the web thickness, dt_w mm²

The web plate buckling coefficient, k_v is determined as follows:

- a. For unstiffened webs with $h/t_w < 260$, $k_v = 5$ except for the stem of tee shapes where $k_v = 1.2$.

- b. For stiffened webs,

$$k_v = 5 + \frac{5}{(a/h)^2}$$

$$= 5 \text{ when } a/h > 3.0 \text{ or } a/h > \left[\frac{260}{h/t_w} \right]^2$$

where:

- a = clear distance between transverse stiffeners, mm
- h = for rolled shapes, the clear distance between flanges less the fillet or corner radii, mm
- = for built-up welded sections, the clear distance between flanges, mm
- = for built-up bolted sections, the distance between fastener lines, mm
- = for tees, the overall depth, mm

507.2.2 Transverse Stiffeners

Transverse stiffeners are not required where $h/t_w \leq 2.46 \sqrt{E/F_y}$, or where the required shear strength is less than or equal to the available shear strength provided in accordance with Section 507.2.1 for $k_v = 5$.

Transverse stiffeners used to develop the available web shear strength, as provided in Section 507.2.1, shall have a moment of inertia about an axis in the web center for stiffener pairs or about the face in contact with the web plate for single stiffeners, which shall not be less than at_w^3/j , where

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \quad (507.2-6)$$

Transverse stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which transverse stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe to the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange. When lateral bracing is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit 1 percent of the total flange force, unless the flange is composed only of angles.

Bolts connecting stiffeners to the girder web shall be spaced not more than 305 mm on center. If intermittent fillet welds are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 250 mm.

507.3 Tension Field Action

507.3.1 Limits on the Use of Tension Field Action

Consideration of tension field action is permitted for flanged members when the web plate is supported on all four sides by flanges or stiffeners. Consideration of tension field action is not permitted for:

1. end panels in all members with transverse stiffeners;
2. members when a/h exceeds 3.0 or $[260/(h/t_w)]^2$;
3. $2A_w/(A_{fc} + A_{ft}) > 2.5$; or
4. h/b_{fc} or $h/b_{ft} > 6.0$

where:

A_{fc} = area of compression flange, mm²
 A_{ft} = area of tension flange, mm²
 b_{fc} = width of compression flange, mm
 b_{ft} = width of tension flange, mm

In these cases, the nominal shear strength, V_n , shall be determined according to the provisions of Section 507.2.

507.3.2 Nominal Shear Strength with Tension Field Action

When tension field action is permitted according to Section 507.3.1, the nominal shear strength, V_n , with tension field action, according to the limit state of tension field yielding, shall be

$$1. \text{ For } h/t_w \leq 1.10 \sqrt{k_v E/F_y}$$

$$V_n = 0.6F_y A_w \quad (507.3-1)$$

$$2. \text{ For } h/t_w > 1.10 \sqrt{k_v E/F_y}$$

$$V_n = 0.6F_y A_w \left(C_v + \frac{1 - C_v}{1.15 \sqrt{1 + (a/h)^2}} \right) \quad (507.3-2)$$

where:

k_v and C_v are as defined in Section 507.2.1.

507.3.3 Transverse Stiffeners

Transverse stiffeners subject to tension field action shall meet the requirements of Section 507.2.2 and the following limitations:

$$1. (b/t)_{st} \leq 0.56 \sqrt{\frac{E}{F_{yst}}}$$

$$2. A_{st} > \frac{F_y}{F_{yst}} \left[0.15 D_s h t_w (1 - C_v) \frac{V_r}{V_c} - 18 t_w^2 \right] \geq 0 \quad (507.3-3)$$

where:

$(b/t)_{st}$ = the width-thickness ratio of the stiffener
 F_{yst} = specified minimum yield stress of the stiffener material, MPa
 C_v = coefficient defined in Section 507.2.1
 D_s = 1.0 for stiffeners in pairs
 = 1.8 for single angle stiffeners
 = 2.4 for single plate stiffeners
 V_r = required shear strength at the location of the stiffener, N
 V_c = available shear strength; $\phi_v V_n$ (LRFD) or V_n/Ω_v (ASD) with V_n as defined in Section 507.3.2, N

507.4 Single Angles

The nominal shear strength, V_n , of a single angle leg shall be determined using Equation (507.2-1) with $C_v = 1.0$, $A_w = bt$ where b = width of the leg resisting the shear force, mm and $k_v = 1.2$.

507.5 Rectangular HSS and Box Members

The nominal shear strength, V_n , of rectangular HSS and box members shall be determined using the provisions of Section 507.2.1 with $A_w = 2ht$ where h for the width resisting the shear force shall be taken as the clear distance between the flanges less the inside corner radius on each side and $t_w = t$ and $k_v = 5$. If the corner radius is not known, h shall be taken as the corresponding outside dimension minus three times the thickness.

507.6 Round HSS

The nominal shear strength, V_n , of round HSS, according to the limit states of shear yielding and shear buckling, is

$$V_n = F_{cr} A_g / 2 \quad (507.6-1)$$

where:

F_{cr} shall be the larger of

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D} \left(\frac{D}{t}\right)^4}} \quad (507.6-2a)$$

and

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^2} \quad (507.6-2b)$$

but shall not exceed $0.6F_y$

A_g = gross area of section based on design wall thickness, mm²

D = outside diameter, mm

L_v = the distance from maximum to zero shear force, mm

t = design wall thickness, equal to 0.93 times the nominal wall thickness for ERW HSS and equal to the nominal thickness for SAW HSS, mm

User Note: The shear buckling equations, Equations 507.6-2a and 507.6-2b, will control for D/t over 100, high strength steels, and long lengths. If the shear strength for standard sections is desired, shear yielding will usually control

507.7 Weak Axis Shear in Singly and Doubly Symmetric Shapes

For singly and doubly symmetric shapes loaded in the weak axis without torsion, the nominal shear strength, V_n , for each shear resisting element shall be determined using Eq. 507.2-1 and Section 507.2.1(2) with $A_w = b_f t_f$ and $k_v = 1.2$.

User Note: For all ASTM A6 W, S, M and HP shapes, when $F_y \leq 345$ MPa, $C_v = 1.0$.

507.8 Beams and Girders with Web Openings

The effect of all web openings on the nominal shear strength of steel and composite beams shall be determined. Adequate reinforcement shall be provided when the required strength exceeds the available strength of the member at the opening.

SECTION 508

DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

This section addresses members subject to axial force and flexure about one or both axes, with or without torsion, and to members subject to torsion only.

508.1 Doubly and Singly Symmetric Members Subject to Flexure and Axial Force

508.1.1 Doubly and Singly Symmetric Members in Flexure and Compression

The interaction of flexure and compression in doubly symmetric members and singly symmetric members for which $0.1 \leq (I_{yc}/I_y) \leq 0.9$, that are constrained to bend about a geometric axis (x and/or y) shall be limited by Eqs. 508.1-1a and 508.1-1b, where I_{yc} is the moment of inertia about the y-axis referred to the compression flange, mm^4 .

User Note: Section 508.2 is permitted to be used in lieu of the provisions of this section.

1. For $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9} \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \quad (508.1-1a)$$

2. For $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \quad (508.1-1b)$$

where:

- P_r = required axial compressive strength, N
- P_c = available axial compressive strength, N
- M_r = required flexural strength, N-mm
- M_c = available flexural strength, N-mm
- x = subscript relating symbol to strong axis bending
- y = subscript relating symbol to weak axis bending

For design according to Section 502.3.3 (LRFD)

- P_r = required axial compressive strength using LRFD load combinations, N
- $P_c = \phi_c P_n$ = design axial compressive strength, determined in accordance with Section 505, N

M_r = required flexural strength using LRFD load combinations, N-mm

$M_c = \phi_b M_n$ = design flexural strength determined in accordance with Section 506, N-mm

ϕ_c = resistance factor for compression = 0.90

ϕ_b = resistance factor for flexure = 0.90

For design according to Section 502.3.4 (ASD)

P_r = required axial compressive strength using ASD load combinations, N

$P_c = P_n/\Omega_c$ = allowable axial compressive strength, determined in accordance with Section 505, N

M_r = required flexural strength using ASD load combinations, N-mm

$M_c = M_n/\Omega_b$ = allowable flexural strength determined in accordance with Section 506, N-mm

Ω_c = safety factor for compression = 1.67

Ω_b = safety factor for flexure = 1.67

508.1.2 Doubly and Singly Symmetric Members in Flexure and Tension

The interaction of flexure and tension in doubly symmetric members and singly symmetric members constrained to bend about a geometric axis (x and/or y) shall be limited by Eqs. 508.1-1a and 508.1-1b,

where

For design according to Section 502.3.3 (LRFD)

P_r = required tensile strength using LRFD load combinations, N

$P_c = \phi_t P_n$ = design tensile strength, determined in accordance with Section 504.2, N

M_r = required flexural strength using LRFD load combinations, N-mm

$M_c = \phi_b M_n$ = design flexural strength determined in accordance with Section 506, N-mm

ϕ_t = resistance factor for tension (see Section 504.2)

ϕ_b = resistance factor for flexure = 0.90

For doubly symmetric members,

C_b in Section 506 may be increased by $\sqrt{1 + \frac{P_u}{P_{ey}}}$ for axial tension that acts concurrently with flexure, where

$$P_{ey} = \frac{\pi^2 EI_y}{L_b^2}$$

For design according to Section 502.3.3 (LRFD)

P_r = required tensile strength using ASD load combinations, N

$P_c = P_n/\Omega_t$ = allowable tensile strength, determined

in accordance with Section 504.2, N
 M_r = required flexural strength using ASD load combinations, N-mm
 $M_c = M_n/\Omega_b$ = allowable flexural strength determined in accordance with Section 506, N-mm
 Ω_t = safety factor for tension (see Section 504.2)
 Ω_b = safety factor for flexure = 1.67

For doubly symmetric members, C_b in Section 506 may be increased by $\sqrt{1 + \frac{1.5P_a}{P_{ey}}}$ for axial tension that acts concurrently with flexure where $P_{ey} = \frac{\pi^2 EI_y}{L_b^2}$

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Eq. 508.1-1a and 508.1-1b.

508.1.3 Doubly Symmetric Members in Single Axis Flexure and Compression

For doubly symmetric members in flexure and compression with moments primarily in one plane, it is permissible to consider the two independent limit states, in-plane instability and out-of-plane buckling or flexural-torsional buckling, separately in lieu of the combined approach provided in Section 508.1.1.

1. For the limit state of in-plane instability, Eqs. 508.1.1 shall be used with P_c , M_r and M_c determined in the plane of bending.
2. For the limit state of out-of-plane buckling

$$\frac{P_r}{P_{co}} + \left(\frac{M_r}{M_{cx}}\right)^2 \leq 1.0 \quad (508.1-2)$$

where

P_{co} = available compressive strength out of the plane of bending, N
 M_{cx} = available flexural-torsional strength for strong axis flexure determined from Section 506, N-mm

If bending occurs only about the weak axis, the moment ratio in Eq. 508.1-2 shall be neglected.

For members with significant biaxial moments ($M_r/M_c \geq 0.05$) in both directions), the provisions of Section 508.1.1 shall be followed.

508.2 Unsymmetric and other Members Subject to Flexure and Axial Force

This section addresses the interaction of flexure and axial stress for shapes not covered in Section 508.1. It is permitted to use the provisions of this Section for any shape in lieu of the provisions of Section 508.1.

$$\left| \frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \right| \leq 1.0 \quad (508.2-1)$$

where

f_a = required axial stress at the point of consideration, MPa
 F_a = available axial stress at the point of consideration, MPa
 f_{bw}, f_{bz} = required flexural stress at the point of consideration, MPa
 F_{bw}, F_{bz} = available flexural stress at the point of consideration, MPa
 w = subscript relating symbol to major principal axis bending
 z = subscript relating symbol to minor principal axis bending

For design according to Section 502.3.3 (LRFD)

f_a = required axial stress using LRFD load combinations, MPa
 F_a = $\phi_c F_{cr}$ = design axial stress, determined in accordance with Section 505 for compression or Section 504.2 for tension, MPa
 f_{bw}, f_{bz} = required flexural stress at the specific location in the cross section using LRFD load combinations, MPa
 F_{bw}, F_{bz} = $\phi_b M_n/S$ = design flexural stress determined in accordance with section 506, MPa. Use the section modulus for the specific location in the cross section and consider the sign of the stress
 ϕ_c = resistance factor for compression = 0.90
 ϕ_t = resistance factor for tension (Section 504.2)
 ϕ_b = resistance factor for flexure = 0.90

For design according to Section 502.3.4 (ASD)

f_a = required axial stress using ASD load combinations, MPa
 F_a = F_{cr}/Ω_c = allowable axial stress determined in accordance with section 505 for compression, or Section 504.2 for tension, MPa

- f_{bw}, f_{bz} = required flexural stress at the specific location in the cross section using ASD load combinations, MPa
- F_{bw}, F_{bz} = $M_n/\Omega_b S$ = allowable flexural stress determined in accordance with Section 506, MPa. Use the section modulus for the specific location in the cross section and consider the sign of the stress
- Ω_c = safety factor for compression = 1.67
- Ω_t = safety factor for tension (Section 504.2)
- Ω_b = safety factor for flexure = 1.67

Eq. 508.2-1 shall be evaluated using the principal bending axes by considering the sense of the flexural stresses at the critical points of the cross section. The flexural terms are either added to or subtracted from the axial term as appropriate. When the axial force is compression, second order effects shall be included according to the provisions of Section 503.

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Eq. 508.2-1.

508.3 Members under Torsion and Combined Torsion, Flexure, Shear and/or Axial Force

508.3.1 Torsional Strength of Round and Rectangular HSS

The design torsional strength, $\phi_T T_n$, and the allowable torsional strength, T_n/Ω_T , for round and rectangular HSS shall be determined as follows:

$$\phi_T = 0.90 \text{ (LRFD)} \quad \Omega_T = 1.67 \text{ (ASD)}$$

The nominal torsional strength, T_n , according to the limit states of torsional yielding and torsional buckling is:

$$T_n = F_{cr} C \quad (508.3-1)$$

where

- C = is the HSS torsional constant
- F_{cr} = shall be determined as follows:

1. For round HSS, F_{cr} shall be the larger of

$$F_{cr} = \frac{1.23E}{\sqrt{\frac{L}{D} \left(\frac{D}{t}\right)^{\frac{5}{4}}}} \quad (508.3-2a)$$

and

$$F_{cr} = \frac{0.60E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \quad (508.3-2b)$$

but shall not exceed $0.6F_y$, where

- L = length of the member, mm
- D = outside diameter, mm

2. For rectangular HSS

- a. For $h/t \leq 2.45 \sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \quad (508.3-3)$$

- b. For $2.45 \sqrt{E/F_y} < h/t \leq 3.07 \sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \left(2.45 \sqrt{E/F_y} \right) / (h/t) \quad (508.3-4)$$

- c. For $3.07 \sqrt{E/F_y} < h/t \leq 260$

$$F_{cr} = 0.458\pi^2 E / (h/t)^2 \quad (508.3-5)$$

User Note: The torsional shear constant, C , may be conservatively taken as:

For a round HSS: $C = \frac{\pi(D-t)^2 t}{2}$

For rectangular HSS:

$$C = 2(B-t)(H-t)t - 4.5(4-\pi)t^3$$

508.3.2 HSS Subject to Combined Torsion, Shear, Flexure and Axial Force

When the required torsional strength, T_r , is less than or equal to 20 percent of the available torsional strength, T_c , the interaction of torsion, shear, flexure and/or axial force for HSS shall be determined by Section 508.1 and the torsional effects shall be neglected. When T_r exceeds, 20 percent of T_c , the interaction of torsion, shear, flexure and/or axial force shall be limited by

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \leq 1.0 \quad (508.3-6)$$

where

For design according to Section 502.3.3 (LRFD)

- P_r = required axial strength using LRFD load combinations, N
- P_c = ϕP_n , design tensile or compressive strength in accordance with Section 504 or 505, N
- M_r = required flexural strength using LRFD load combinations, N-mm
- M_c = $\phi_b M_n$, design flexural strength in accordance with Section 506, N-mm
- V_r = required shear strength using LRFD load combinations, N
- V_c = $\phi_v V_n$, design shear strength in accordance with Section 507, N
- T_r = required torsional strength using LRFD load combinations, N-mm
- T_c = $\phi_T T_n$, design torsional strength in accordance with Section 508.3.1, N-mm

For design according to Section 502.3.4 (ASD)

- P_r = required axial strength using ASD load combinations, N
- P_c = P_n/Ω , allowable tensile or compressive strength in accordance with Section 504 or 505, N
- M_r = required flexural strength using ASD load combinations determined in accordance with Section 502.5, N-mm
- M_c = M_n/Ω_b , allowable flexural strength in accordance with Section 506, N-mm
- V_r = required shear strength using ASD load combinations, N
- V_c = V_n/Ω_v , allowable shear strength in accordance with Section 507, N
- T_r = required torsional strength using ASD load combinations, N-mm
- T_c = T_n/Ω_T , allowable torsional strength in accordance with Section 508.3.1, N-mm

508.3.3 Strength of Non-HSS Members under Torsion and Combined Stress

The design torsional strength, $\phi_T F_n$, and the allowable torsional strength, F_n/Ω_T , for non-HSS members shall be the lowest value obtained according to the limit states of yielding under normal stress, shear yielding under shear stress, or buckling, determined as follows:

$$\phi_T = 0.90 \text{ (LRFD)} \quad \Omega_T = 1.67 \text{ (ASD)}$$

1. For the limit state of yielding under normal stress

$$F_n = F_y \quad (508.3-7)$$

2. For the limit state of shear yielding under shear stress

$$F_n = 0.6 F_y \quad (508.3-8)$$

3. or the limit state of buckling

$$F_n = F_{cr} \quad (508.3-9)$$

where

F_{cr} = buckling stress for the section as determined by analysis, MPa.

Some constrained local yielding is permitted adjacent to areas that remain elastic

SECTION 509

DESIGN OF COMPOSITE MEMBERS

509.1 General Provisions

In determining load effects in members and connections of a structure that includes composite members, consideration shall be given to the effective sections at the time each increment of load is applied. The design, detailing and material properties related to the concrete and reinforcing steel portions of composite construction shall comply with the reinforced concrete and reinforcing bar design specifications stipulated by applicable building code. In the absence of a building code, the provisions in Chapter 4 shall apply.

509.1.1 Nominal Strength of Composite Sections

Two methods are provided for determining the nominal strength of composite sections: the plastic stress distribution method and the strain-compatibility method.

The tensile strength of the concrete shall be neglected in the determination of the nominal strength of composite members.

509.1.1.1 Plastic Stress Distribution Method

For the plastic stress distribution method, the nominal strength shall be computed assuming that steel components have reached a stress of F_y in either tension or compression and concrete components in compression have reached a stress of $0.85f'_c$. For round HSS filled with concrete, a stress of $0.95f'_c$ is permitted to be used for concrete components in uniform compression to account for the effects of concrete confinement.

509.1.1.2 Strain-Compatibility Method

For the strain compatibility method, a linear distribution of strains across the section shall be assumed, with the maximum concrete compressive strain equal to 0.003 mm/mm. The stress-strain relationships for steel and concrete shall be obtained from tests or from published results for similar materials.

User Note: The strain compatibility method should be used to determine nominal strength for irregular sections and for cases where the steel does not exhibit elasto-plastic behavior. General guidelines for the strain-compatibility method for encased columns are given in AISC Design Guide 6 and Sections 410.3 and 410.4 of this Code.

509.1.2 Material Limitations

Concrete and steel reinforcing bars in composite systems shall be subject to the following limitations.

1. For the determination of the available strength, concrete shall have a compressive strength f'_c of not less than 21 MPa nor more than 70 MPa for normal weight concrete and not less than 21 MPa nor more than 42 MPa for lightweight concrete.

User Note: Higher strength concrete materials may be used for stiffness calculations but may not be relied upon for strength calculations unless justified by testing or analysis.

2. The specified minimum yield stress of structural steel and reinforcing bars used in calculating the strength of a composite column shall not exceed 525 MPa.

Higher material strengths are permitted when their use is justified by testing or analysis.

User Note: Additional reinforced concrete material limitations are specified in Chapter 4.

509.1.3 Shear Connectors

Shear connectors shall be headed steel studs not less than four stud diameters in length after installation, or hot-rolled steel channels. Shear stud design values shall be taken as per Sections 509.2.1.7 and 509.3.2.4 (2). Stud connectors shall conform to the requirements of Section 501.3.6. Channel connectors shall conform to the requirements of Section 501.3.1.

509.2 Axial Members

This section applies to two types of composite axial members: encased and filled sections.

509.2.1 Encased Composite Columns**509.2.1.1.1 Limitations**

To qualify as an encased composite column, the following limitations shall be met:

1. The cross-sectional area of the steel core shall comprise at least 1 percent of the total composite cross section.
2. Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals. The minimum transverse reinforcement shall be at least 6 mm² per mm of tie spacing.
3. The minimum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr} , shall be 0.004, where ρ_{sr} is given by:

$$\rho_{sr} = \frac{A_{sr}}{A_g} \quad (509.2-1)$$

where

A_{sr} = area of continuous reinforcing bars, mm²
 A_g = gross area of composite member, mm²

509.2.1.1.2 Compressive Strength

The design compressive strength, $\phi_c P_n$, and allowable compressive strength, P_n/Ω_c , for axially loaded encased composite columns shall be determined for the limit state of flexural buckling based on column slenderness as follows:

$$\phi_c = 0.75(LRFD) \quad \Omega_c = 2.00(ASD)$$

1. When $P_e \geq 0.44P_o$

$$P_n = P_o \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right] \quad (509.2-2)$$

2. When $P_e < 0.44P_o$

$$P_n = 0.877P_e \quad (509.2-3)$$

where

$$P_o = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f'_c \quad (509.2-4)$$

$$P_e = \pi^2 (EI_{eff}) / (KL)^2 \quad (509.2-5)$$

where

A_s = area of the steel section, mm²

A_c = area of concrete, mm²

A_{sr} = area of continuous reinforcing bars, mm²

E_c = modulus of elasticity of concrete

$$(0.043 w_c^{1.5} \sqrt{f'_c}, \text{MPa})$$

E_s = modulus of elasticity of steel = 200,000 MPa

f'_c = specified compressive strength of concrete, MPa

F_y = specified minimum yield stress of steel section, MPa

F_{yr} = specified minimum yield stress of reinforcing bars, MPa

I_c = moment of inertia of the concrete section, mm⁴

I_s = moment of inertia of steel shape, mm⁴

I_{sr} = moment of inertia of reinforcing bars, mm⁴

K = the effective length factor determined in accordance with Section 502

L = laterally unbraced length of the member, mm

w_c = weight of concrete per unit volume

$$(1500 \leq w_c \leq 2500 \frac{\text{kg}}{\text{m}^3})$$

where

EI_{eff} = effective stiffness of composite section, N-mm²

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \quad (509.2-6)$$

where

$$C_1 = 0.1 + 2 \left(\frac{A_s}{A_c + A_s} \right) \leq 0.3 \quad (509.2-7)$$

509.2.1.3.3 Tensile Strength

The design tensile strength, $\phi_t P_n$, and allowable tensile strength, P_n/Ω_t , for encased composite columns shall be determined for the limit state of yielding as

$$P_n = A_s F_y + A_{sr} F_{yr} \quad (509.2-8)$$

$$\phi_t = 0.90(LRFD) \quad \Omega_t = 1.67(ASD)$$

509.2.1.4.4 Shear Strength

The available shear strength shall be calculated based on either the shear strength of the steel section alone as specified in Section 507 plus the shear strength provided by tie reinforcement, if present, or the shear strength of the reinforced concrete portion alone.

User Note: The nominal shear strength of tie reinforcement may be determined as $A_{st}F_{yr}(d/s)$ where A_{st} is the area of tie reinforcement, d is the effective depth of the concrete section, and s is the spacing of the tie reinforcement. The shear capacity of reinforced concrete may be determined according to Chapter 4 Section 411.

509.2.1.5.5 Load Transfer

Loads applied to axially loaded encased composite columns shall be transferred between the steel and concrete in accordance with the following requirements:

1. When the external force is applied directly to the steel section, shear connectors shall be provided to transfer the required shear force, V' , as follows:

$$V' = V(1 - A_s F_y / P_o) \quad (509.2-9)$$

where

V = required shear force introduced to column, N

A_s = area of steel cross section, mm²

P_o = nominal axial compressive strength without consideration of length effects, N

2. When the external force is applied directly to the concrete encasement, shear connectors shall be provided to transfer the required shear force, V' , as follows:

$$V' = V(A_s F_y / P_o) \quad (509.2-10)$$

3. When load is applied to the concrete of an encased composite column by direct bearing the design bearing strength, $\phi_B P_p$, and the allowable bearing strength, P_p / Ω_B , of the concrete shall be:

$$P_p = 1.7 f'_c A_B \quad (509.2-11)$$

$$\phi_B = 0.65(LRFD) \quad \Omega_B = 2.31(ASD)$$

where

A_B = loaded area of concrete, mm²

509.2.1.6.6 Detailing Requirements

At least four continuous longitudinal reinforcing bars shall be used in encased composite columns. Transverse reinforcement shall be spaced at the smallest of 16 longitudinal bar diameters, 48 tie bar diameters or 0.5 times the least dimension of the composite section. The encasement shall provide at least 40 mm of clear cover to the reinforcing steel.

Shear connectors shall be provided to transfer the required shear force specified in Section 509.2.1.5. The shear connectors shall be distributed along the length of the member at least a distance of 2.5 times the depth of the encased composite column above and below the load transfer region. The maximum connector spacing shall be 400mm.

Connectors to transfer axial load shall be placed on at least two faces of the steel shape in a configuration symmetrical about the steel shape axes.

If the composite cross section is built up from two or more encased steel shapes, the shapes shall be interconnected with lacing, tie plates, batten plates or similar components to prevent buckling of individual shapes due to loads applied prior to hardening of the concrete.

509.2.1.7.7 Strength of Stud Shear Connectors

The nominal strength of one stud shear connector embedded in solid concrete is:

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq A_{sc} F_u \quad (509.2-12)$$

where

A_{sc} = cross-sectional area of stud shear connector, mm²

F_u = specified minimum tensile strength of a stud shear connector, MPa

509.2.2 Filled Composite Columns

509.2.2.1 Limitations

To qualify as a filled composite column the following limitations shall be met:

1. The cross-sectional area of the steel HSS shall comprise at least 1 percent of the total composite cross section.
2. The maximum b/t ratio for a rectangular HSS used as a composite column shall be equal to $2.26\sqrt{E/F_y}$. Higher ratios are permitted when their use is justified

by testing or analysis.

3. The maximum D/t ratio for a round HSS filled with concrete shall be $0.15 E/F_y$. Higher ratios are permitted when their use is justified by testing or analysis.

509.2.2.2 Compressive Strength

The design compressive strength, $\phi_c P_n$, and allowable compressive strength, P_n/Ω_c , for axially loaded filled composite columns shall be determined for the limit state of flexural buckling based on Section 509.2.1.2 with the following modifications:

$$P_o = A_s f_y + A_{sr} F_{yr} + C_2 A_c f'_c \quad (509.2-13)$$

$C_2 = 0.85$ for rectangular sections and 0.95 for circular sections

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \quad (509.2-14)$$

$$C_3 = 0.6 + 2 \left(\frac{A_s}{A_c + A_s} \right) \leq 0.9 \quad (509.2-15)$$

509.2.2.3 Tensile Strength

The design tensile strength, $\phi_t P_n$, and allowable tensile strength, P_n/Ω_t , for filled composite columns shall be determined for the limit state of yielding as:

$$P_n = A_s F_y + A_{sr} F_{yr} \quad (509.2-16)$$

$$\phi_t = 0.90(LRFD) \quad \Omega_t = 1.67(ASD)$$

509.2.2.4 Shear Strength

The available shear strength shall be calculated based on either the shear strength of the steel section alone as specified in Section 507 or the shear strength of the reinforced concrete portion alone.

509.2.2.5 Load Transfer

Loads applied to filled composite columns shall be transferred between the steel and concrete. When the external force is applied either to the steel section or to the concrete infill, transfer of force from the steel section to the concrete core is required from direct bond interaction, shear connection or direct bearing. The force transfer mechanism providing the largest nominal strength may be used. These force transfer mechanisms shall not be superimposed.

When load is applied to the concrete of an encased or filled composite column by direct bearing the design bearing strength, $\phi_B P_p$, and the allowable bearing strength, P_p/Ω_B of the concrete shall be:

$$P_p = 1.7 f'_c A_B \quad (509.2-17)$$

$$\phi_B = 0.65(LRFD) \quad \Omega_B = 2.31(ASD)$$

where

A_B = the loaded area, mm^2

509.2.2.6 Detailing Requirements

Where required, shear connectors transferring the required shear force shall be distributed along the length of the member at least a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS both above and below the load transfer region. The maximum connector spacing shall be 400 mm.

509.3 Flexural Members

509.3.1 General

509.3.1.1 Effective Width

The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline, each of which shall not exceed:

- one-eighth of the beam span, center-to-center of supports;
- one-half the distance to the centerline of the adjacent beam; or
- the distance to the edge of the slab.

509.3.1.2 Shear Strength

The available shear strength of composite beams with shear connectors shall be determined based upon the properties of the steel section alone in accordance with Section 507. The available shear strength of concrete-encased and filled composite members shall be determined based upon the properties of the steel section alone in accordance with Section 507 or based upon the properties of the concrete and longitudinal steel reinforcement.

509.3.1.3 Strength During Construction

When temporary shores are not used during construction, the steel section alone shall have adequate strength to support all loads applied prior to the concrete attaining 75 percent of its specified strength f'_c . The available flexural strength of the steel section shall be determined according to Section 506.

509.3.2 Strength of Composite Beams with Shear Connectors

509.3.2.1 Positive Flexural Strength

The design positive flexural strength, $\phi_b M_n$, and the allowable positive flexural strength, M_n/Ω_b , shall be determined for the limit state of yielding as follows:

$$\phi_b = 0.90(LRFD) \quad \Omega_b = 1.67(ASD)$$

1. For $h/t_w \leq 3.76 \sqrt{E/F_y}$,

M_n , shall be determined from the plastic stress distribution on the composite section for the limit state of yielding (plastic moment).

User Note: All current ASTM A6 W, S and HP shapes satisfy the limit given in Section 509.3.2.2.1(1) for $F_y \leq 345 \text{ MPa}$.

2. For $h/t_w > 3.76 \sqrt{E/F_y}$,

M_n , shall be determined from the superposition of elastic stresses, considering the effects of shoring, for the limit state of yielding (yield moment).

509.3.2.2 Negative Flexural Strength

The design negative flexural strength, $\phi_b M_n$, and the allowable negative flexural strength, M_n/Ω_b , shall be determined for the steel section alone, in accordance with the requirements of Section 506.

Alternatively, the available negative flexural strength shall be determined from the plastic stress distribution on the composite section, for the limit state of yielding (plastic moment), with

$$\phi_b = 0.90(LRFD) \quad \Omega_b = 1.67(ASD)$$

provided that:

1. The steel beam is compact and is adequately braced according to Section 506.
2. Shear connectors connect the slab to the steel beam in the negative moment region.
3. The slab reinforcement parallel to the steel beam, within the effective width of the slab, is properly developed.

509.3.2.3 Strength of Composite Beams with Formed Steel Deck

1. General

The available flexural strength of composite construction consisting of concrete slabs on formed steel deck connected to steel beams shall be determined by the applicable portions of Section 509.3.2a and 509.3.2b, with the following requirements:

- a. This section is applicable to decks with nominal rib height not greater than 75 mm. The average width of concrete rib or haunch, w_r , shall be not less than 50 mm, but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.
 - b. The concrete slab shall be connected to the steel beam with welded stud shear connectors 20 mm or less in diameter (AWS D1.1). Studs shall be welded either through the deck or directly to the steel cross section. Stud shear connectors, after installation, shall extend not less than 40 mm above the top of the steel deck and there shall be at least 12 mm of concrete cover above the top of the installed studs.
 - c. The slab thickness above the steel deck shall be not less than 50 mm.
 - d. Steel deck shall be anchored to all supporting members at a spacing not to exceed 450 mm. Such anchorage shall be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other devices specified by the designer.
2. Deck Ribs Oriented Perpendicular to Steel Beam
Concrete below the top of the steel deck shall be neglected in determining composite section properties and in calculating A_c for deck ribs oriented perpendicular to the steel beams.

3. Deck Ribs Oriented Parallel to Steel Beam

- Concrete below the top of the steel deck may be included in determining composite section properties and shall be included in calculating A_c .

Formed steel deck ribs over supporting beams may be split longitudinally and separated to form a concrete haunch.

When the nominal depth of steel deck is 40 mm or greater, the average width, w_r , of the supported haunch or rib shall be not less than 50 mm for the first stud in the transverse row plus four stud diameters for each additional stud.

509.3.2.4 Shear Connectors

1. Load Transfer for Positive Moment

The entire horizontal shear at the interface between the steel beam and the concrete slab shall be assumed to be transferred by shear connectors, except for concrete-encased beams as defined in Section 509.3. For composite action with concrete subject to flexural compression, the total horizontal shear force, V' , between the point of maximum positive moment and the point of zero moment shall be taken as the lowest value according to the limit states of concrete crushing, tensile yielding of the steel section, or strength of the shear connectors:

- Concrete crushing

$$V' = 0.85f'_c A_c \quad (509.3-1a)$$

- Tensile yielding of the steel section

$$V' = F_y A_s \quad (509.3-1b)$$

- Strength of shear connectors

$$V' = \Sigma Q_n \quad (509.3-1c)$$

where

- A_c = area of concrete slab within effective width, mm²
 A_s = area of steel cross section, mm²
 ΣQ_n = sum of nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment, N

2. Load Transfer for Negative Moment

In continuous composite beams where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel beam, the total horizontal shear force between the point of maximum negative moment and the point of zero moment shall be taken as the lower value according to the limit states of yielding of the steel reinforcement in the slab, or strength of the shear connectors:

- Tensile yielding of the slab reinforcement

$$V' = A_r F_{yr} \quad (509.3-2a)$$

where

- A_r = area of adequately developed longitudinal reinforcing steel within the effective width of the concrete slab, mm²
 F_{yr} = specified minimum yield stress of the reinforcing steel, MPa

- Strength of shear connectors

$$V' = \Sigma Q_n \quad (509.3-2b)$$

3. Strength of Stud Shear Connectors

The nominal strength of one stud shear connector embedded in solid concrete or in a composite slab is

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq R_g R_p A_{sc} F_u \quad (509.3-3)$$

where

- A_{sc} = cross-sectional area of stud shear connector, mm²
 E_c = modulus of elasticity of concrete
 $= (0.043 w_c^{1.5} \sqrt{f'_c}, \text{MPa})$
 F_u = specified minimum tensile strength of a stud shear connector, MPa
 R_g = 1.0; (a) for one stud welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for any number of studs welded in a row directly to the steel shape; (c) for any number of studs welded in a row through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth ≥ 1.5
 $= 0.85$; (a) for two studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for one stud welded through steel deck with

- the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth < 1.5
- R_p = 0.7 for three or more studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape
- R_p = 1.0 for studs welded directly to the steel shape (in other words, not through steel deck or sheet) and having a haunch detail with not more than 50 percent of the top flange covered by deck or sheet steel closures
- = 0.75; (a) for studs welded in a composite slab with the deck oriented perpendicular to the beam and $e_{mid-ht} \geq 50$ mm; (b) for studs welded through steel deck, or steel sheet used as girder filler material, and embedded in a composite slab with the deck oriented parallel to the beam
- = 0.6 for studs welded in a composite slab with deck oriented perpendicular to the beam and
- $e_{mid-ht} < 50$ mm
- e_{mid-ht} = distance from the edge of stud shank to the steel deck web, measured at mid-height of the deck rib, and in the load bearing direction of the stud (in other words, in the direction of maximum moment for a simply supported beam), mm
- w_c = weight of concrete per unit volume ($1500 \leq w_c \leq 2500 \text{ kg/m}^3$)

User Note: The table below presents values for R_g and R_p for several cases

Condition	R_g	R_p
No Decking*	1.0	1.0
Decking oriented parallel to the steel shape		
$w_r/h_r \geq 1.5$	1.0	0.75
$w_r/h_r < 1.5$	0.85**	0.75
Decking oriented perpendicular to the steel shape		
Number of studs occupying the same decking rib		
1	1.0	0.6†
2	0.85	0.6†
3 or more	0.70	0.6†

h_r = nominal rib height, mm

w_r = average width of concrete rib or haunch as defined Section 509.1.3c, mm

* to qualify as "no decking," stud shear connectors shall be welded directly to the steel

shape and no more than 50 percent of the top flange of the steel shape maybe covered by decking or steel sheet, such as girder filler material.

** for a single stud

† this value maybe increased to 0.75 when $e_{mid-ht} \geq 50$ mm

4. Strength of Channel Shear Connectors

The nominal strength of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f'_c E_c} \quad (509.3-4)$$

where

t_f = flange thickness of channel shear connector, mm

t_w = web thickness of channel shear connector, mm

L_c = length of channel shear connector, mm

The strength of the channel shear connector shall be developed by welding the channel to the beam flange for a force equal to Q_n , considering eccentricity on the connector.

5. Required Number of Shear Connectors

The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the horizontal shear force as determined in Sections 509.3.2.4(1) and 509.3.2.4(2) divided by the nominal strength of one shear connector as determined from Section 509.3.2.4(3) or Section 509.3.2.4(4).

6. Shear Connector Placement and Spacing

Shear connectors required on each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment, unless otherwise specified. However, the number of shear connectors placed between any concentrated load and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the concentrated load point.

Shear connectors shall have at least 25 mm of lateral concrete cover, except for connectors installed in the ribs of formed steel decks. The diameter of studs shall not be

greater than 2.5 times the thickness of the flange to which they are welded, unless located over the web. The minimum center-to-center spacing of stud connectors shall be six diameters along the longitudinal axis of the supporting composite beam and four diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks oriented perpendicular to the steel beam the minimum center-to-center spacing shall be four diameters in any direction. The maximum center-to-center spacing of shear connectors shall not exceed eight times the total slab thickness.

509.4 Flexural Strength of Concrete-Encased and Filled Members

The nominal flexural strength of concrete-encased and filled members shall be determined using one of the following methods:

- a. The superposition of elastic stresses on the composite section, considering the effects of shoring, for the limit state of yielding (yield moment),

where

$$\phi_b = 0.90(LRFD) \quad \Omega_b = 1.67(ASD)$$

- b. The plastic stress distribution on the steel section alone, for the limit state of yielding (plastic moment), where

$$\phi_b = 0.90(LRFD) \quad \Omega_b = 1.67(ASD)$$

- c. If shear connectors are provided and the concrete meets the requirements of Section 509.1.2, the nominal flexural strength shall be computed based upon the plastic stress distribution on the composite section or from the strain-compatibility method,

where

$$\phi_b = 0.85(LRFD) \quad \Omega_b = 1.76(ASD)$$

509.5 Combined Axial Force and Flexure

The interaction between axial forces and flexure in composite members shall account for stability as required by Section 503. The design compressive strength, $\phi_c P_n$, and allowable compressive strength, P_n/Ω_c , and the design flexural strength, $\phi_b M_n$, and allowable flexural strength, M_n/Ω_b , are determined as follows:

$$\phi_c = 0.75(LRFD) \quad \Omega_c = 2.00(ASD)$$

$$\phi_b = 0.90(LRFD) \quad \Omega_b = 1.67(ASD)$$

1. The nominal strength of the cross section of a composite member subjected to combined axial compression and flexure shall be determined using either the plastic stress distribution method or the strain-compatibility method.
2. To account for the influence of length effects on the axial strength of the member, the nominal axial strength of the member shall be determined by Section 509.2 with P_o taken as the nominal axial strength of the cross section determined in Section 509.4 (1) above.

509.6 Special Cases

When composite construction does not conform to the requirements of Section 509.1 through Section 509.4, the strength of shear connectors and details of construction shall be established by testing.

SECTION 510

DESIGN OF CONNECTIONS

510.1 General Provisions

510.1.1 Design Basis

The design strength, ϕR_n , and the allowable strength R_n/Ω , of connections shall be determined in accordance with the provisions of this section and the provisions of Section 502.

The required strength of the connections shall be determined by structural analysis for the specified design loads, consistent with the type of construction specified, or shall be a proportion of the required strength of the connected members when so specified herein.

Where the gravity axes of intersecting axially loaded members do not intersect at one point, the effects of eccentricity shall be considered.

510.1.2 Simple Connections

Simple connections of beams, girders, or trusses shall be designed as flexible and are permitted to be proportioned for the reaction shears only, except as otherwise indicated in the design documents. Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic, but self-limiting deformation in the connection is permitted to accommodate the end rotation of a simple beam.

510.1.3 Moment Connections

End connections of restrained beams, girders, and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections. Response criteria for moment connections are provided in Section 502.3.6.2.

User Note: See Section 503 and Appendix A-7 for analysis requirements to establish the required strength and stiffness for design of connections.

510.1.4 Compression Members with Bearing Joints

1. When columns bear on bearing plates or are finished to bear at splices, there shall be sufficient connectors to hold all parts securely in place.
2. When compression members other than columns are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for either (a) or (b) below. It is permissible to use the less severe of the two conditions:

- a. An axial tensile force of 50 percent of the required compressive strength of the member; or
- b. The moment and shear resulting from a transverse load equal to 2 percent of the required compressive strength of the member. The transverse load shall be applied at the location of the splice exclusive of other loads that act on the member. The member shall be taken as pinned for the determination of the shears and moments at the splice.

User Note: All compression joints should also be proportioned to resist any tension developed by the load combinations stipulated in Section 502.2.

510.1.5 Splices in Heavy Sections

When tensile forces due to applied tension or flexure are to be transmitted through splices in heavy sections, as defined in Section 501.3.1.3 and 501.3.1.4, by complete joint-penetration groove (CJP) welds, material notch-toughness requirements as given in Section 501.3.1.3 and 501.3.1.4, weld access hole details as given in Section 510.1.6 and thermal cut surface preparation and inspection requirements as given in Section 513.2.2 shall apply. The foregoing provision is not applicable to splices of elements of built-up shapes that are welded prior to assembling the shape.

User Note: CJP groove welded splices of heavy sections can exhibit detrimental effects of weld shrinkage. Members that are sized for compression that are also subject to tensile forces may be less susceptible to damage from shrinkage if they are spliced using PJP groove welds on the flanges and fillet-welded web plates or using bolts for some or all of the splice.

510.1.6 Beam Copes and Weld Access Holes

All weld access holes required to facilitate welding operations shall have a length from the toe of the weld preparation not less than $1\frac{1}{2}$ times the thickness of the material in which the hole is made. The height of the access hole shall be $1\frac{1}{2}$ times the thickness of the material with the access hole, t_w , but not less than 25 mm nor does it need to exceed 50 mm. The access hole shall be detailed to provide room for weld backing as needed.

For sections that are rolled or welded prior to cutting, the edge of the web shall be sloped or curved from the surface of the flange to the reentrant surface of the access hole. In hot-rolled shapes, and built-up shapes with CJP groove welds that join the web-to-flange, all beam copes and weld access holes shall be free of notches and sharp reentrant corners. No arc of the weld access hole shall have a radius less than 10 mm.

In built-up shapes with fillet or partial-joint-penetration groove welds that join the web-to-flange, all beam copes and weld access holes shall be free of notches and sharp reentrant corners. The access hole shall be permitted to terminate perpendicular to the flange, providing the weld is terminated at least a distance equal to the weld size away from the access hole.

For heavy sections as defined in Section 501.3.1.3 and Section 501.3.1.4, the thermally cut surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods prior to deposition of splice welds. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle methods.

510.1.7 Placement of Welds and Bolts

Groups of welds or bolts at the ends of any member which transmit axial force into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically loaded single angle, double angle, and similar members.

510.1.8 Bolts in Combination with Welds

Bolts shall not be considered as sharing the load in combination with welds, except that shear connections with any grade of bolts permitted by Section 501.3.3 installed in standard holes or short slots transverse to the direction of the load are permitted to be considered to share the load with longitudinally loaded fillet welds. In such connections the available strength of the bolts shall not be taken as greater than 50 percent of the available strength of bearing-type bolts in the connection.

In making welded alterations to structures, existing rivets and high strength bolts tightened to the requirements for slip-critical connections are permitted to be utilized for carrying loads present at the time of alteration and the welding need only provide the additional required strength.

510.1.9 High-Strength Bolts in Combination with Rivets

In both new work and alterations, in connections designed as slip-critical connections in accordance with the provisions of Section 510.3, high-strength bolts are permitted to be considered as sharing the load with existing rivets.

510.1.10 Limitations on Bolted and Welded Connections

Pretensioned joints, slip-critical joints or welds shall be used for the following connections:

1. Column splices in all multi-story structures over 38 m in height.
2. Connections of all beams and girders to columns and any other beams and girders on which the bracing of columns is dependent in structures over 38 m in height.
3. In all structures carrying cranes of over 50 kN capacity: roof truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports.
4. Connections for the support of machinery and other live loads that produce impact or reversal of load

Snug-tightened joints or joints with ASTM A307 bolts shall be permitted except where otherwise specified.

510.2 Welds

All provisions of AWS D1.1 apply under this Specification, with the exception that the provisions of the listed NSCP Specification Sections apply under this Specification in lieu of the cited AWS provisions as follows:

NSCP Structural Steel Section 510.1.6 in lieu of AWS D1.1 Section 5.17.1

NSCP Structural Steel Section 510.2.2a in lieu of AWS D1.1 Section 2.3.2

NSCP Structural Steel Table 510.2.2 in lieu of AWS D1.1 Table 2.1

NSCP Structural Steel Table 510.2.5 in lieu of AWS D1.1 Table 2.3

NSCP Structural Steel Appendix A-3, Table A-3.1 in lieu of AWS D1.1 Table 2.4

NSCP Structural Steel Section 502.3.9 and Appendix A-3 in lieu of AWS D1.1 Section 2, Part C

NSCP Structural Steel Section 513.2 in lieu of AWS D1.1 Sections 5.15.4.3 and 5.15.4.4

510.2.1 Groove Welds

510.2.1.1 Effective Area

The effective area of groove welds shall be considered as the length of the weld times the effective throat thickness.

The effective throat thickness of a complete-joint-penetration (CJP) groove weld shall be the thickness of the thinner part joined.

Table 510.2.1 Effective Throat of Partial-Joint-Penetration Groove Welds

Welding Process	Welding Position F (flat), H (horiz.), V (vert.), OH (overhead)	Groove Type (AWS D1.1, Figure 3.3)	Effective Throat
Shielded Metal Arc (SMAW)	All	J or U Groove 60° V	Depth of Groove
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	All		
Submerged Arc (SAW)	F		
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	F, H	45° Bevel	Depth of Groove
Shielded Metal Arc (SMAW)	All	45° Bevel	Depth of Groove Minus 3 mm
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	V, OH	45° Bevel	Depth of Groove Minus 3 mm

The effective throat thickness of a partial-joint-penetration (PJP) groove weld shall be as shown in Table 510.2.1.

User Note: The effective throat size of a partial-joint-penetration groove weld is dependent on the process used and the weld position. The contract documents should either indicate the effective throat required or the weld strength required, and the fabricator should detail the joint based on the weld process and position to be used to weld the joint.

The effective weld size for flare groove welds, when filled flush to the surface of a round bar, a 90° bend in a formed section, or rectangular HSS shall be as shown in Table 510.2.2, unless other effective throats are demonstrated by tests. The effective size of flare groove welds filled less than flush shall be as shown in Table 510.2.2, less the greatest perpendicular dimension measured from a line flush to the base metal surface to the weld surface.

Table 510.2.2 Effective Weld Sizes of Flare Groove Welds

Welding Process	Flare Bevel Groove ^[a]	Flare V Groove
GMAW and FCAW-G	$\frac{5}{8}R$	$\frac{3}{4}R$
SMAW and FCAW-S	$\frac{5}{16}R$	$\frac{5}{8}R$
SAW	$\frac{5}{16}R$	$\frac{1}{2}R$

^[a]For Flare Bevel Groove with $R < 10 \text{ mm}$ use only reinforcing fillet weld on filled flush joint. General Note: R =radius of joint surface (can be assumed to be $2t$ for HSS), mm

Table 510.2.3 Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Material Thickness of Thinner Part Joined, mm	Minimum Effective Throat Thickness, ^[a] mm.
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6
Over 19 to 38	8
Over 38 to 57	10
Over 57 to 150	13
Over 150	16

^[a]See Table 510.2.1.

Larger effective throat thicknesses than those in Table 510.2.2 are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.

510.2.1.2 Limitations

The minimum effective throat thickness of a partial-joint-penetration groove weld shall not be less than the size required to transmit calculated forces nor the size shown in Table 510.2.3. Minimum weld size is determined by the thinner of the two parts joined.

Table 510.2.4 Minimum Size of Fillet Welds

Material Thickness of Thinner Part Joined, mm	Minimum size of Fillet weld, ^[a] mm.
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6
Over 19	8

^[a]Leg Dimension of fillet welds. Single pass welds must be used.
Note: See Section 510.2.2b for maximum size of fillet welds.

510.2.2 Fillet Welds

510.2.2.1 Effective Area

The effective area of a fillet weld shall be the effective length multiplied by the effective throat. The effective throat of a fillet weld shall be the shortest distance from the root to the face of the diagrammatic weld. An increase in effective throat is permitted if consistent penetration beyond the root of the diagrammatic weld is demonstrated by tests using the production process and procedure variables.

For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

510.2.2.2 Limitations

The minimum size of fillet welds shall be not less than the size required to transmit calculated forces, nor the size as shown in Table 510.2.4. These provisions do not apply to fillet weld reinforcements of partial- or complete-joint-penetration groove welds.

The maximum size of fillet welds of connected parts shall be:

1. Along edges of material less than 6 mm thick, not greater than the thickness of the material.

2. Along edges of material 6 mm or more in thickness, not greater than the thickness of the material minus 2 mm, unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 2 mm provided the weld size is clearly verifiable.

The minimum effective length of fillet welds designed on the basis of strength shall be not less than four times the nominal size, or else the size of the weld shall be considered not to exceed $\frac{1}{4}$ of its effective length. If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. For the effect of longitudinal fillet weld length in end connections upon the effective area of the connected member, see Section 504.3.3.

For end-loaded fillet welds with a length up to 100 times the leg dimension, it is permitted to take the effective length equal to the actual length. When the length of the end-loaded fillet weld exceeds 100 times the weld size, the effective length shall be determined by multiplying the actual length by the reduction factor, β ,

$$\beta = 1.2 - 0.002(L/w) \leq 1.0 \quad (510.2-1)$$

where

L = actual length of end-loaded weld, mm
 w = weld leg size, mm

When the length of the weld exceeds 300 times the leg size, the value of β shall be taken as 0.60.

Intermittent fillet welds are permitted to be used to transfer calculated stress across a joint or faying surfaces when the required strength is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of built-up members. The effective length of any segment of intermittent fillet welding shall be not less than four times the weld size, with a minimum of 38 mm.

In lap joints, the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than 25 mm. Lap joints joining plates or bars subjected to axial stress that utilize transverse fillet welds only shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

Fillet weld terminations are permitted to be stopped short or extend to the ends or sides of parts or be boxed except as limited by the following:

1. For lap joints in which one connected part extends beyond an edge of another connected part that is subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from that edge.
2. For connections where flexibility of the outstanding elements is required, when end returns are used, the length of the return shall not exceed four times the nominal size of the weld nor half the width of the part.
3. Fillet welds joining transverse stiffeners to plate girder webs 19 mm thick or less shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of stiffeners are welded to the flange.
4. Fillet welds that occur on opposite sides of a common plane shall be interrupted at the corner common to both welds.

User Note: Fillet weld terminations should be located approximately one weld size from the edge of the connection to minimize notches in the base metal. Fillet welds terminated at the end of the joint, other than those connecting stiffeners to girder webs, are not a cause for correction.

Fillet welds in holes or slots are permitted to be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap, subject to the provisions of Section 510.2. Fillet welds in holes or slots are not to be considered plug or slot welds.

510.2.3 Plug and Slot Welds

510.2.3.1 Effective Area

The effective shearing area of plug and slot welds shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

510.2.3.2 Limitations

Plug or slot welds are permitted to be used to transmit shear in lap joints or to prevent buckling of lapped parts and to join component parts of built-up members.

The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus 8 mm, rounded to the next larger even mm, nor greater than the minimum diameter plus 3 mm or $2\frac{1}{4}$ times the thickness of the weld.

The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus 8 mm rounded to the next larger even mm, nor shall it be larger than $2\frac{1}{4}$ times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot. The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

The thickness of plug or slot welds in material 16 mm or less in thickness shall be equal to the thickness of the material. In material over 16 mm thick, the thickness of the weld shall be at least one-half the thickness of the material but not less than 16 mm.

510.2.4 Strength

The design strength, ϕR_n , and the allowable strength, R_n/Ω , of welds shall be the lower value of the base material and the weld metal strength determined according to the limit states of tensile rupture, shear rupture or yielding as follows:

For the base metal

$$R_n = F_{BM} A_{BM} \quad (510.2-2)$$

For the weld metal

$$R_n = F_w A_w \quad (510.2-3)$$

where

F_{BM} = nominal strength of the base metal per unit area, MPa

F_w = nominal strength of the weld metal per unit area, MPa

A_{BM} = cross-sectional area of the base metal, mm²

A_w = effective area of the weld, mm²

The values of ϕ , Ω , F_{BM} , F_w and limitations thereon are given in Table 510.2.5.

Alternatively, for fillet welds loaded in-plane the design strength, ϕR_n and the allowable strength, R_n/Ω , of welds is permitted to be determined as follows:

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

1. For a linear weld group loaded in-plane through the center of gravity

$$R_n = F_w A_w \quad (510.2-4)$$

where

$$F_w = 0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \theta) \quad (510.2-5)$$

and

F_{EXX} = electrode classification number, MPa

θ = angle of loading measured from the weld longitudinal axis, degrees

A_w = effective area of the weld, mm²

User Note: A linear weld group is one in which all elements are in a line or are parallel.

2. For weld elements within a weld group that are loaded in-plane and analyzed using an instantaneous center of rotation method, the components of the nominal strength, R_{nx} and R_{ny} , are permitted to be determined as follows:

$$R_{nx} = \Sigma F_{wix} A_{wi} \quad R_{ny} = \quad (510.2-6)$$

where

A_{wi} = effective area of weld throat of any i^{th} weld element, mm²

$$F_{wi} = 0.60 F_{EXX} (1.0 + 0.50 \sin 1.5 \theta) f(p) \quad (510.2-7)$$

$$f(p) = [p(1.9 - 0.9p)]^{0.3} \quad (510.2-8)$$

F_{wi} = nominal stress in any i^{th} weld element, MPa

F_{wix} = x component of F_{wi}

F_{wiy} = y component of stress, F_{wi}

p = Δ_i / Δ_m , ratio of element i deformation to its deformation at maximum stress

w = weld leg size, mm

r_{crit} = distance from instantaneous center of rotation to weld element with minimum Δ_u / r_i ratio, mm

Δ_i = deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i , mm

$$= r_i \Delta_u / r_{crit}$$

Δ_m = $0.209(\theta + 2)^{-0.32} w$, deformation of weld element at maximum stress, mm

Δ_u = $1.087(6)^{-0.65} w \leq 0.17w$, deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, mm

1. For fillet weld groups concentrically loaded and consisting of elements that are oriented both longitudinally and transversely to the direction of applied load, the combined strength, R_n , of the fillet weld group shall be determined as the greater of

$$R_n = R_{wl} + R_{wt} \quad (510.2-9a)$$

or

$$R_n = 0.85 R_{wl} + 1.5 R_{wt} \quad (510.2-9b)$$

where

R_{wl} = the total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table 510.2.5, N

R_{wt} = the total nominal strength of transversely loaded fillet welds, as determined in accordance with Table 510.2.5 without the alternate in Section 510.2.4(1), N

510.2.5 Combination of Welds

If two or more of the general types of welds (groove, fillet, plug, slot) are combined in a single joint, the strength of each shall be separately computed with reference to the axis of the group in order to determine the strength of the combination.

510.2.6 Filler Metal Requirements

The choice of electrode for use with complete-joint-penetration groove welds subject to tension normal to the effective area shall comply with the requirements for matching filler metals given in AWS D1.1/D.1M.

User Note: The following User Note Table summarizes the AWS D1. 1 provision for matching filler metals. Other restrictions exist. For a complete list of base metals and prequalified matching filler metals see AWS D1.1/D.1M, Table 3.1.

Base Metal	Matching Filler Metal
A36 $\leq \frac{3}{4}$ in. thick	60 & 70 ksi Electrodes
A36 $> \frac{3}{4}$ in. A572 (Gr. 50 & 55) A913 (Gr. 50) A588* A992 A1011 A1018 A91 3 (Gr. 60 & 65)	SMAW: E7015, E7016, E7018, E7028 Other processes: 70 ksi electrodes 80 ksi electrodes
* For corrosion resistance and color similar to the base see AWS D1.1, Section 3.7.3 Notes: 1. Filler metal shall meet the requirements of AWS A5.1, A5.5, A5.17, A5.18, A5.20, A5.23, A5.28 and A5.29. 2. In joints with base metals of different strengths use either a filler metal that matches the higher strength base metal or a filler metal that matches the lower strength and produces a low hydrogen deposit.	

Table 510.2.5 Available Strength of Welded Joints, N

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{BM} or F_w) N	Effective Area (A_{BM} or A_w) mm ²	Required Filler Metal Strength Level ^{[a][b]}
COMPLETE-JOINT-PENETRATION GROOVE WELDS					
Tension Normal to weld axis	Strength of the joint is controlled by the base metal				Matching filler metal shall be used. For T and corner joints with backing left in place, notch tough filler metal is required. See Section 510.2.6.
Compression Normal to weld axis	Strength of the joint is controlled by the base metal				Filler metal with a strength level equal to or one strength level less than matching filler metal is permitted.
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				Filler metal with a strength level equal to or less than matching filler metal is permitted.
Shear	Strength of the joint is controlled by the base metal				Matching filler metal shall be used. ^[c]

Table 510.2.5 Available Strength of Welded Joints, N (continuation)

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{BM} or F_w) N	Effective Area (A_{BM} or A_w) mm^2	Required Filler Metal Strength Level ^{[a][b]}	
PARTIAL-JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE VEE GROOVE AND FLARE BEVEL GROOVE WELDS						
Tension Normal to weld axis	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See Section 510.4	Filler metal with a strength level equal to or less than matching filler metal is permitted	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60F_{EXX}$	See Section 510.2.1a		
Compression Column to base Plate and column splices designed per Section 510.1.4(1)	Compressive stress need not be considered in design of welds joining the parts.					
Compression Connections of members designed to bear other than columns as described in Section 510.1.4(2)	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See Section 510.4		
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60F_{EXX}$	See Section 510.2.1a		
Compression Connections not finished-to-bear	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See Section 510.4		
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.90F_{EXX}$	See Section 510.2.1a		
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.					
Shear	Base	Governed by Section 510.4				
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$	See Section 510.2.2a		
PLUG AND SLOT WELDS						
Shear	Base	Governed by 510.4				
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$	See Section 510.2.3a		

(a) For matching weld metal see AWS D1 .1, Section 3.3.

(b) Filler metal with a strength level one strength level greater than matching is permitted.

(c) Filler metals with a strength level less than matching may be used for groove welds between the webs and flanges of built-up sections transferring shear loads, or in applications where high restraint is a concern. In these applications, the weld joint shall be detailed and the weld shall be designed using the thickness of the material as the effective throat, $\phi = 0.80$, $\Omega = 1.88$ and $0.60F_{EXX}$ as the nominal strength

(d) Alternatively, the provisions of Section 510.2.4(1) are permitted provided the deformation compatibility of the various weld elements is considered. Alternatively, Sections 510.2.4(2) and 510.2.4(3) are special applications of Section 510.2.4(1) that provide for deformation compatibility

Filler metal with a specified Charpy V-Notch (CVN) toughness of 27 J at 4 °C shall be used in the following joints:

1. Complete-joint-penetration groove welded T and corner joints with steel backing left in place, subject to tension normal to the effective area, unless the joints are designed using the nominal strength and resistance factor or safety factor as applicable for a partial-joint-penetration groove weld.
2. Complete-joint-penetration groove welded splices subject to tension normal to the effective area in heavy sections as defined in Section 501.3.1.3 and 501.3.1.4.

The manufacturer's Certificate of Conformance shall be sufficient evidence of compliance.

510.2.7 Mixed Weld Metal

When Charpy V-Notch toughness is specified, the process consumables for all weld metal, tack welds, root pass and subsequent passes deposited in a joint shall be compatible to ensure notch-tough composite weld metal.

510.3 Bolts and Threaded Parts

510.3.1 High-Strength Bolts

Use of high-strength bolts shall conform to the provisions of the Specification for Structural Joints Using ASTM A325 or A490 Bolts, hereafter referred to as the RCSC Specification, as approved by the Research Council on Structural Connections, except as otherwise provided in this Specification.

When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale. All ASTM A325 or A325M and A490 or A490M bolts shall be tightened to a bolt tension not less than that given in Table 510.3.1, except as noted below.

Table 510.3.1 Minimum Bolt Pretension, kN*

Bolt Size, mm	A325M Bolts	A490M Bolts
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334
M30	326	408
M36	475	595

*Equal to 0.70 times the minimum tensile strength of bolts, rounded off to nearest kN, as specified in ASTM specifications for A325M and A490M bolts with UNC threads.

Except as permitted below, installation shall be assured by any of the following methods: turn-of-nut method, a direct tension indicator, calibrated wrench or alternative design bolt.

Bolts are permitted to be installed to only the snug-tight condition when used in

1. bearing-type connections.
2. tension or combined shear and tension applications, for ASTM A325 or A325M bolts only, where loosening or fatigue due to vibration or load fluctuations are not design considerations.

The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact. Bolts to be tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings.

When ASTM A490 or A490M bolts over 25 mm in diameter are used in slotted or oversized holes in external plies, a single hardened washer conforming to ASTM F436, except with 8 mm minimum thickness, shall be used in lieu of the standard washer.

User Note: Washer requirements are provided in the RCSC Specification, Section 6.

Table 510.3.2 Nominal Stress of Fasteners and Threaded Parts, MPa

Description of Fasteners	Nominal Tensile Stress, F_{nt} , MPa	Nominal Shear Stress in Bearing-Type Connections, F_{nv} , MPa
A307 bolts	310 _{[a][b]}	165 _{[b][c][f]}
A325 or A325M bolts, when threads are not excluded from shear planes	620 _[e]	372 _[f]
A325 or A325M bolts, when threads are excluded from shear planes	620 _[e]	457 _[f]
A490 or A490M bolts, when threads are not excluded from shear planes	780 _[e]	457 _[f]
A490 or A490M bolts, when threads are excluded from shear planes	780 _[e]	579 _[f]
Threaded parts meeting the requirements of Section 501.3.4, when threads are not excluded from shear planes	$0.75F_u^{[a][d]}$	$0.450F_u$
Threaded parts meeting the requirements of Section 501.3.4, when threads are excluded from shear planes	$0.75F_u^{[a][d]}$	$0.563F_u$
[a] Subject to the requirements of Appendix A-3 [b] For A307 bolts the tabulated values shall be reduced by 1 percent for each 2 mm over 5 diameters of length in the grip [c] Threads permitted in shear planes [d] The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, A_d, which shall be larger than the nominal body area of the rod before upsetting times F_y [e] For A325 or A325M and A490 or A490M bolts subject to tensile fatigue loading, see Appendix A-3 [f] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 1270 mm, tabulated values shall be reduced by 20 percent		

In slip-critical connections in which the direction of loading is toward an edge of a connected part, adequate available bearing strength shall be provided based upon the applicable requirements of Section 510.3.10.

When bolt requirements cannot be provided by ASTM A325 and A325M, F1852, or A490 and A490M bolts because of requirements for lengths exceeding 12 diameters or diameters exceeding 38 mm, bolts or threaded rods conforming to ASTM A354 Gr. BC, A354 Gr. BD, or A449 are permitted to be used in accordance with the provisions for threaded rods in Table 510.3.2.

When ASTM A354 Gr. BC, A354 Gr. BD, or A449 bolts and threaded rods are used in slip-critical connections, the bolt geometry including the head and nut(s) shall be equal to or (if larger in diameter) proportional to that provided by ASTM A325 and A325M, or ASTM A490 and A490M bolts. Installation shall comply with all applicable requirements of the RCSC Specification with modifications as required for the increased diameter and/or length to provide the design pretension.

510.3.2. Size and Use of Holes

The maximum sizes of holes for bolts are given in Table 510.3.3, except that larger holes, required for tolerance on location of anchor rods in concrete foundations, are permitted in column base details.

Table 510.3.3 Nominal Hole Dimensions, mm

Bolt Diameter	Hole Dimensions			
	Standard (Dia)	Oversize (Dia)	Short-Slot (Width x Length)	Long-Slot (Width x Length)
M16	18	20	18 x 22	18 x 40
M20	22	24	22 x 26	22 x 50
M22	24	28	24 x 30	24 x 55
M24	27 ^(a)	30	27 x 32	27 x 60
M27	30	35	30 x 37	30 x 67
M30	33	38	33 x 40	33 x 75
≥M36	$d+3$	$d+8$	$(d+3) \times (d+10)$	$(d+3) \times 2.5 d$

^(a) Clearance provided allows the use of a M30 bolt if desirable.

Standard holes or short-slotted holes transverse to the direction of the load shall be provided in accordance with the provisions of this specification, unless oversized holes, short-slotted holes parallel to the load or long-slotted holes are approved by the engineer-of-record. Finger shims up to 6 mm are permitted in slip-critical connections designed on the basis of standard holes without reducing the nominal shear strength of the fastener to that specified for slotted holes.

Oversized holes are permitted in any or all plies of slip-critical connections, but they shall not be used in bearing-type connections. Hardened washers shall be installed over oversized holes in an outer ply.

Short-slotted holes are permitted in any or all plies of slip-critical or bearing-type connections. The slots are permitted without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

Long-slotted holes are permitted in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes are permitted without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 8 mm thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

510.3.3 Minimum Spacing

The distance between centers of standard, oversized, or slotted holes, shall not be less than $2\frac{2}{3}$ times the nominal diameter, d , of the fastener; a distance of $3d$ is preferred.

510.3.4 Minimum Edge Distance

The distance from the center of a standard hole to an edge of a connected part in any direction shall not be less than either the applicable value from Table 510.3.4, or as required in Section 510.3.10. The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole to an edge of a connected part plus the applicable increment C_2 from Table 510.3.5.

User Note: The edge distances in Tables 510.3.4 are minimum edge distances based on standard fabrication practices and workmanship tolerances. The appropriate provisions of Sections 510.3.10 and 510.4 must be satisfied.

Table 510.3.4 Minimum Edge Distance, [a] mm, from Center of Standard Hole [b] to Edge of Connected Part

Bolt Diameter (mm)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges ^[c]
16	28	22
20	34	26
22	38 ^[d]	28
24	42 ^[d]	30
27	48	34
30	52	38
36	64	46
Over 36	$1.75d$	$1.25d$

^[a] Lesser edge distances are permitted to be used provided provisions of Section 510.3.10, as appropriate, are satisfied.

^[b] For oversized or slotted holes, see Table 510.3.5

^[c] All edge distances in this column are permitted to be reduced 3 mm when the hole is at a point where required strength does not exceed 25 percent of the maximum strength in the element.

^[d] These are permitted to be 32 mm at the ends of beam connection angles and shear end plates.

Table 510.3.5 Values of Edge Distance Increment C_2 , mm

Nominal Diameter of Fastener (mm)	Oversized Holes	Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots ^[a]	
≤ 22	2	3	$0.75d$	0
24	3	3		
≥ 27	3	5		

^[a] When length of slot is less than maximum allowable (see Table 510.3.3), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

510.3.5 Maximum Spacing and Edge Distance

The maximum distance from the center of any bolt or rivet to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 150 mm. The longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates shall be as follows:

1. For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 300 mm.
2. For unpainted members of weathering steel subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or 180 mm.

510.3.6 Tension and Shear Strength of Bolts and Threaded Parts

The design tension or shear strength, ϕR_n , and the allowable tension or shear strength, R_n/Ω of a snug-tightened or pretensioned high-strength bolt or threaded part shall be determined according to the limit states of tensile rupture and shear rupture as follows:

$$R_n = F_n A_b \quad (510.3-1)$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

where

F_n = nominal tensile stress F_{nt} , or shear stress, F_{nv} from Table 510.3.2, MPa

A_b = nominal unthreaded body area of bolt or threaded part (for upset rods, see footnote d , Table 510.3.2), mm²

The required tensile strength shall include any tension resulting from prying action produced by deformation of the connected parts.

510.3.7 Combined Tension and Shear in Bearing-Type Connections

The available tensile strength of a bolt subjected to combined tension and shear shall be determined according to the limit states of tension and shear rupture as follows:

$$R_n = F'_{nt} A_b \quad (510.3-2)$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

where

F'_{nt} = nominal tensile stress modified to include the effects of shearing stress, MPa

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt} \text{ (LRFD)} \quad (510.3-3a)$$

$$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \leq F_{nt} \text{ (ASD)} \quad (510.3-3b)$$

F_{nt} = nominal tensile stress from Table 510.3.2, MPa

F_{nv} = nominal shear stress from Table 510.3.2, MPa

f_v = the required shear stress, MPa

The available shear stress of the fastener shall equal or exceed the required shear strength per unit area, f_v .

User Note: Note that when the required stress, f , in either shear or tension, is less than or equal to 30% of the corresponding available stress, the effects of combined stress need not be investigated. Also note that Eqs. 510.3-3a and 510.3-3b can be rewritten so as to find a nominal shear stress, F'_{nv} , as a function of the required tensile stress, f_t .

510.3.8 High-Strength Bolts in Slip-Critical Connections

High-strength bolts in slip-critical connections are permitted to be designed to prevent slip either as a serviceability limit state or at the required strength limit state. The connection must also be checked for shear strength in accordance with Sections 510.3.6 and 510.3.7 and bearing strength in accordance with Sections 510.3.1 and 510.3.10.

Slip-critical connections shall be designed as follows, unless otherwise designated by the engineer-of-record. Connections with standard holes or slots transverse to the direction of the load shall be designed for slip as a serviceability limit state. Connections with oversized holes or slots parallel to the direction of the load shall be designed to prevent slip at the required strength level.

The design slip resistance, ϕR_n , and the allowable slip resistance, R_n/Ω , shall be determined for the limit state of slip as follows:

$$R_n = \mu D_u h_{sc} T_b N_s \quad (510.3-4)$$

For connections in which prevention of slip is a serviceability limit state

$$\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

For connections designed to prevent slip at the required strength level

$$\phi = 0.85 \text{ (LRFD)} \quad \Omega = 1.76 \text{ (ASD)}$$

where

μ = mean slip coefficient for Class A or B surfaces, as applicable, or as established by tests

= 0.35 for Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel and hot-dipped galvanized and roughened surfaces)

= 0.50 for Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

D_u = 1.13; a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension. The use of other values may be approved by the engineer-of-record.

h_{sc} = hole factor determined as follows:

(a) For standard size holes $h_{sc} = 1.00$

(b) For oversized and short-slotted holes $h_{sc} = 0.85$

(c) For long-slotted holes $h_{sc} = 0.70$

N_s = number of slip planes

T_b = minimum fastener tension given in Table 510.3.1, kN

User Note: There are special cases where, with oversize holes and slots parallel to the load, the movement possible due to connection slip could cause a structural failure. Resistance and safety factors are provided for connections where slip is prevented until the required strength load is reached.

Design loads are used for either design method and all connections must be checked for strength as bearing-type connections.

510.3.9 Combined Tension and Shear in Slip-Critical Connections

When a slip-critical connection is subjected to an applied tension that reduces the net clamping force, the available slip resistance per bolt, from Section 510.3.8, shall be multiplied by the factor, k_s , as follows:

$$k_s = 1 - \frac{T_u}{D_u T_b N_b} \quad (LRFD) \quad (510.3-5a)$$

$$k_s = 1 - \frac{1.5 T_a}{D_u T_b N_b} \quad (ASD) \quad (510.3-5b)$$

where

N_b = number of bolts carrying the applied tension

T_a = tension force due to ASD load combinations, kN

T_b = minimum fastener tension given in Table 510.3.1, kN.

T_u = tension force due to LRFD load combinations, kN

510.3.10 Bearing Strength at Bolt Holes

The available bearing strength, ϕR_n and R_n/Ω , at bolt holes shall be determined for the limit state of bearing as follows:

$$\phi = 0.75 (LRFD) \quad \Omega = 2.00 (ASD)$$

1. For a bolt in a connection with standard, oversized, and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force:

- a. when deformation at the bolt hole at service load is a design consideration

$$R_n = 1.2 L_c t F_u \leq 2.4 d t F_u \quad (510.3-6a)$$

- b. when deformation at the bolt hole at service load is not a design consideration

$$R_n = 1.5 L_c t F_u \leq 3.0 d t F_u \quad (510.3-6b)$$

2. For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force:

$$R_n = 1.0 L_c t F_u \leq 2.0 d t F_u \quad (510.3-6c)$$

3. For connections made using bolts that pass completely through an unstiffened box member or HSS, see Section 510.7 and Eq. 510.7-1,

where

d = nominal bolt diameter, mm

F_u = specified minimum tensile strength of the connected material, MPa

L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, mm

t = thickness of connected material, mm

For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts.

Bearing strength shall be checked for both bearing-type and slip-critical connections. The use of oversized holes and short- and long-slotted holes parallel to the line of force is restricted to slip-critical connections per Section 510.3.2.

510.3.11 Special Fasteners

The nominal strength of special fasteners other than the bolts presented in Table 510.3.2 shall be verified by tests.

510.3.12 Tension Fasteners

When bolts or other fasteners in tension are attached to an unstiffened box or HSS wall, the strength of the wall shall be determined by rational analysis.

510.4 Affected Elements of Members and Connecting Elements

This section applies to elements of members at connections and connecting elements, such as plates, gussets, angles, and brackets.

510.4.1 Strength of Elements in Tension

The design strength, ϕR_n , and the allowable strength, R_n/Ω , of affected and connecting elements loaded in tension shall be the lower value obtained according to the limit states of tensile yielding and tensile rupture.

1. For tensile yielding of connecting elements:

$$R_n = F_y A_g \quad (510.4-1)$$

$$\phi = 0.75(LRFD) \quad \Omega = 1.67(ASD)$$

2. For tensile rupture of connecting elements:

$$R_n = F_u A_e \quad (510.4-2)$$

$$\phi = 0.75(LRFD) \quad \Omega = 2.00(ASD)$$

where

A_e = effective net area as defined in Section 504.3.3, mm²; for bolted splice plates,

$$A_e = A_n \leq 0.85 A_g$$

510.4.2 Strength of Elements in Shear

The available shear yield strength of affected and connecting elements in shear shall be the lower value obtained according to the limit states of shear yielding and shear rupture:

1. For shear yielding of the element:

$$R_n = 0.60 F_y A_g \quad (510.4-3)$$

$$\phi = 1.00(LRFD) \quad \Omega = 1.50(ASD)$$

2. For shear rupture of the element:

$$R_n = 0.6 F_u A_{nv} \quad (510.4-4)$$

$$\phi = 0.75(LRFD) \quad \Omega = 2.00(ASD)$$

where

A_{nv} = net area subject to shear, mm²

510.4.3 Block Shear Strength

The available strength for the limit state of block shear rupture along a shear failure path or path(s) and a perpendicular tension failure path shall be taken as

$$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} \quad (510.4-5)$$

$$\phi = 0.75(LRFD) \quad \Omega = 2.00(ASD)$$

where

A_{gv} = gross area subject to shear, mm²

A_{nt} = net area subject to tension, mm²

A_{nv} = net area subject to shear, mm²

Where the tension stress is uniform, $U_{bs} = 1$; where the tension stress is nonuniform, $U_{bs} = 0.5$.

User Note: The cases where U_{bs} must be taken equal to 0.5

510.4.4 Strength of Elements in Compression

The available strength of connecting elements in compression for the limit states of yielding and buckling shall be determined as follows.

For $KL/r \leq 25$

$$P_n = F_y A_g \quad (510.4-6)$$

$$\phi = 0.90(LRFD) \quad \Omega = 1.67(ASD)$$

For $KL/r \leq 25$ the provisions of Section 505 apply.

510.5 Fillers

In welded construction, any filler 6 mm or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than 6 mm thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plus the thickness of the filler plate.

When a bolt that carries load passes through fillers that are equal to or less than 6 mm thick, the shear strength shall be used without reduction. When a bolt that carries load passes through fillers that are greater than 6 mm thick, one of the following requirements shall apply:

1. For fillers that are equal to or less than 20 mm thick, the shear strength of the bolts shall be multiplied by the factor $[1 - 0.0154(t - 6)]$, where t is the total thickness of the fillers up to 20 mm

2. The fillers shall be extended beyond the joint and the filler extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross section of the connected element and the fillers;
3. The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or
4. The joint shall be designed to prevent slip at required strength levels in accordance with Section 510.3.8.

510.6 Splices

Groove-welded splices in plate girders and beams shall develop the nominal strength of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice.

510.7 Bearing Strength

The design bearing strength, ϕR_n , and the allowable bearing strength, R_n/Ω , of surfaces in contact shall be determined for the limit state of bearing (local compressive yielding) as follows:

$$\phi = 0.75(LRFD) \quad \Omega = 2.00(ASD)$$

The nominal bearing strength, R_n , is defined as follows for the various types of bearing:

1. For finished surfaces, pins in reamed, drilled, or bored holes, and ends of fitted bearing stiffeners:

$$R_n = 1.8F_y A_{pb} \quad (510.7-1)$$

where

F_y = specified minimum yield stress, MPa

A_{pb} = projected bearing area, mm²

2. For expansion rollers and rockers

- a. When $d \leq 635 \text{ mm}$

$$R_n = 1.2(F_y - 90) l_b d / 20 \quad (510.7-2)$$

- b. When $d > 635 \text{ mm}$

$$R_n = 30.2(F_y - 90) l_b \sqrt{d} / 20 \quad (510.7-3)$$

where

d = diameter, mm

l_b = length of bearing, mm

510.8 Column Bases and Bearing on Concrete

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, the design bearing strength, $\phi_c P_p$, and the allowable bearing strength, P_p/Ω_c , for the limit state of concrete crushing are permitted to be taken as follows:

$$\phi_c = 0.65(LRFD) \quad \Omega_c = 2.31(ASD)$$

The nominal bearing strength, P_p , is determined as follows:

1. On the full area of a concrete support:

$$P_p = 0.85f'_c A_1 \quad (510.8-1)$$

2. On less than the full area of a concrete support:

$$P_p = 0.85f'_c A_1 \sqrt{A_2/A_1} \leq 1.7f'_c A_1 \quad (510.8-2)$$

where

A_1 = area of steel concentrically bearing on a concrete support, mm²

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm²

f'_c = ? ??

510.9 Anchor Rods and Embedments

Anchor rods shall be designed to provide the required resistance to loads on the completed structure at the base of columns including the net tensile components of any bending moment that may result from load combinations stipulated in Section 502.2. The anchor rods shall be designed in accordance with the requirements for threaded parts in Table 510.3.2.

Larger oversized and slotted holes are permitted in base plates when adequate bearing is provided for the nut by using ASTM F844 washers or plate washers.

User Note: The permitted hole sizes and corresponding washer dimensions are given in the AISC Manual of Steel Construction

When horizontal forces are present at column bases, these forces should, where possible, be resisted by bearing against concrete elements or by shear friction between the column base plate and the foundation. When anchor rods are designed to resist horizontal force the base plate hole size, the anchor rod setting tolerance, and the horizontal movement of the column shall be considered in the design.

User Note: See Chapter 4 for embedment design and for shear friction design. See OSHA for special erection requirements for anchor rods.

510.10 Flanges and Webs with Concentrated Forces

This section applies to single- and double-concentrated force applied normal to the flange(s) of wide flange sections and similar built-up shapes. A single-concentrated force can be either tensile or compressive. Double-concentrated forces are one tensile and one compressive and form a couple on the same side of the loaded member.

When the required strength exceeds the available strength as determined for the limit states listed in this section, stiffeners and/or doublers shall be provided and shall be sized for the difference between the required strength and the available strength for the applicable limit state. Stiffeners shall also meet the design requirements in Section 510.10.8. Doublers shall also meet the design requirement in Section 510.10.9.

User Note: See Appendix A-6.3 for requirements for the ends of cantilever members.

Stiffeners are required at unframed ends of beams in accordance with the requirements of Section 510.10.7.

510.10.1 Flange Local Bending

This section applies to tensile single-concentrated forces and the tensile component of double-concentrated forces.

The design strength, ϕR_n , and the allowable strength, R_n/Ω , for the limit state of flange local bending shall be determined as follows:

$$R_n = 6.25t_f^2 F_{yf} \quad (510.10-1)$$

$$\phi = 0.90(LRFD) \quad \Omega = 1.67(ASD)$$

where

F_{yf} = specified minimum yield stress of the flange, MPa

t_f = thickness of the loaded flange, mm

If the length of loading across the member flange is less than $0.15b_f$, where b_f is the member flange width, Eq. 510.10-1 need not be checked.

When the concentrated force to be resisted is applied at a distance from the member end that is less than $10t_f$, R_n shall be reduced by 50 percent.

When required, a pair of transverse stiffeners shall be provided.

510.10.2 Web Local Yielding

This section applies to single-concentrated forces and both components of double-concentrated forces.

The available strength for the limit state of web local yielding shall be determined as follows:

$$\phi = 1.00(LRFD) \quad \Omega = 1.50(ASD)$$

The nominal strength, R_n , shall be determined as follows:

1. When the concentrated force to be resisted is applied at a distance from the member end that is greater than the depth of the member d ,

$$R_n = (5k + N)F_{yw}t_w \quad (510.10-2)$$

2. When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member d ,

$$R_n = (2.5k + N)F_{yw}t_w \quad (510.10-3)$$

where

k = distance from outer face of the flange to the web toe of the fillet, mm

F_{yw} = specified minimum yield stress of the web, MPa

N = length of bearing (not less than k for end beam reactions), mm

t_w = web thickness, mm

When required, a pair of transverse stiffeners or a doubler plate shall be provided.

510.10.3 Web Local Crippling

This section applies to compressive single-concentrated forces or the compressive component of double-concentrated forces.

The available strength for the limit state of web local crippling shall be determined as follows:

$$\phi = 0.75(\text{LRFD}) \quad \Omega = 2.00(\text{ASD})$$

The nominal strength, R_n , shall be determined as follows:

1. When the concentrated compressive force to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$:

$$R_n = 0.80t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \quad (510.10-4)$$

2. When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than $d/2$:

- a. For $N/d \leq 0.2$

$$R_n = 0.40t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \quad (510.10-5a)$$

- b. For $N/d > 0.2$

$$R_n = 0.40t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \quad (510.10-5b)$$

where

d = overall depth of the member, mm
 t_f = flange thickness, mm

When required, a transverse stiffener, or pair of transverse stiffeners, or a doubler plate extending at least one-half the depth of the web shall be provided.

510.10.4 Web Sidesway Buckling

This section applies only to compressive single-concentrated forces applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force.

The available strength of the web shall be determined as follows:

$$\phi = 0.85(\text{LRFD}) \quad \Omega = 1.76(\text{ASD})$$

The nominal strength, R_n , for the limit state of web sidesway buckling shall be determined as follows:

1. If the compression flange is restrained against rotation:

- a. For $(h/t_w)/(l/b_f) \leq 2.3$

$$R_n = \frac{C_r t_w^3 t_f}{h^2} \left[1 + 0.4 \left(\frac{h/t_w}{l/b_f} \right)^3 \right] \quad (510.10-6)$$

- b. For $(h/t_w)/(l/b_f) > 2.3$, the limit state of web sidesway buckling does not apply.

When the required strength of the web exceeds the available strength, local lateral bracing shall be provided at the tension flange or either a pair of transverse stiffeners or a doubler plate shall be provided.

2. If the compression flange is not restrained against rotation:

- a. For $(h/t_w)/(l/b_f) \leq 1.7$

$$R_n = \frac{C_r t_w^3 t_f}{h^2} \left[0.4 \left(\frac{h/t_w}{l/b_f} \right)^3 \right] \quad (510.10-7)$$

- b. For $(h/t_w)/(l/b_f) > 1.7$, the limit state of web sidesway buckling does not apply.

When the required strength of the web exceeds the available strength, local lateral bracing shall be provided at both flanges at the point of application of the concentrated forces.

In Eqs. 510.10-6 and 510.10-7, the following definitions apply:

- b_f = flange width, mm
 C_r = 6.62×10^6 MPa when $M_u < M_y$ (LRFD) or $1.5M_a < M_y$ (ASD) at the location of the force
 = 3.31×10^6 MPa when $M_u \geq M_y$ (LRFD) or $1.5M_a \geq M_y$ (ASD) at the location of the force
 h = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of fasteners or the clear distance between flanges when welds are used for built-up shapes, mm.
 l = largest laterally unbraced length along either flange at the point of load, mm
 t_f = flange thickness, mm
 t_w = web thickness, mm

User Note: For determination of adequate restraint, refer to Appendix A-6.

510.10.5 Web Compression Buckling

This section applies to a pair of compressive single-concentrated forces or the compressive components in a pair of double-concentrated forces, applied at both flanges of a member at the same location.

The available strength for the limit state of web local buckling shall be determined as follows:

$$R_n = \frac{24t_w^3 \sqrt{EF_{yw}}}{h} \quad (510.10-8)$$

$$\phi = 0.90(LRFD) \quad \Omega = 1.67(ASD)$$

When the pair of concentrated compressive forces to be resisted is applied at a distance from the member end that is less than $d/2$, R_n shall be reduced by 50 percent.

When required, a single transverse stiffener, a pair of transverse stiffeners, or a doubler plate extending the full depth of the web shall be provided.

510.10.6 Web Panel Zone Shear

This section applies to double-concentrated forces applied to one or both flanges of a member at the same location. The available strength of the web panel zone for the limit state of shear yielding shall be determined as follows:

$$\phi = 0.90(LRFD) \quad \Omega = 1.67(ASD)$$

The nominal strength, R_n shall be determined as follows:

- When the effect of panel-zone deformation on frame stability is not considered in the analysis:

$$\text{a. For } P_r \leq 0.4P_c$$

$$R_n = 0.60F_y d_c t_w \quad (510.10-9)$$

$$\text{b. For } P_r > 0.4P_c$$

$$R_n = 0.60F_y d_c t_w \left(1.4 - \frac{P_r}{P_c} \right) \quad (510.1-10)$$

- When frame stability, including plastic panel-zone deformation, is considered in the analysis:

$$\text{a. For } P_r \leq 0.75P_c$$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w} \right) \quad (510.10-11)$$

$$\text{b. For } P_r > 0.75P_c$$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w} \right) \left(1.9 - \frac{1.2P_r}{P_c} \right) \quad (510.1-12)$$

In Eqs. 510.10-9 through 510.10-12, the following definitions apply:

- A = column cross-sectional area, mm²
 b_{cf} = width of column flange, mm
 d_b = beam depth, mm
 d_c = column depth, mm
 F_y = specified minimum yield stress of the column web, MPa
 P_c = P_y , N (LRFD)
 P_c = $0.6P_y$, N (ASD)
 P_r = required strength, N
 P_y = $F_y A$, axial yield strength of the column, N
 t_{cf} = thickness of the column flange, mm
 t_w = column web thickness, mm

When required, doubler plate(s) or a pair of diagonal stiffeners shall be provided within the boundaries the rigid connection whose webs lie in a common plane.

See Section 510.10.9 for doubler plate design requirements.

510.10.7 Unframed Ends of Beams and Girders

At unframed ends of beams and girders not otherwise restrained against rotation about their longitudinal axes, a pair of transverse stiffeners, extending the full depth of the web, shall be provided.

510.10.8 Additional Stiffener Requirements for Concentrated Forces

Stiffeners required to resist tensile concentrated forces shall be designed in accordance with the requirements of Section 504 and welded to the loaded flange and the web. The welds to the flange shall be sized for the difference between the required strength and available limit state strength. The stiffener to web welds shall be sized to transfer to the web the algebraic difference in tensile force at the ends of the stiffener.

Stiffeners required to resist compressive concentrated forces shall be designed in accordance with the requirements in Sections 505.6.2 and 510.4.4 and shall either bear on or be welded to the loaded flange and welded to the web. The welds to the flange shall be sized for the difference between the required strength and the applicable limit state strength. The weld to the web shall be sized to transfer to the web the algebraic difference in compression force at the ends of the stiffener. For fitted bearing stiffeners, see Section 510.7.

Transverse full depth bearing stiffeners for compressive forces applied to a beam or plate girder flange(s) shall be designed as axially compressed members (columns) in accordance with the requirements of Sections 505.6.2 and 510.4.4.

The member properties shall be determined using an effective length of $0.75h$ and a cross section composed of two stiffeners and a strip of the web having a width of $25t_w$ at interior stiffeners and $25t_w$ at the ends of members. The weld connecting full depth bearing stiffeners to the web shall be sized to transmit the difference in compressive force at each of the stiffeners to the web.

Transverse and diagonal stiffeners shall comply with the following additional criteria:

1. The width of each stiffener plus one-half the thickness of the column web shall not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.
2. The thickness of a stiffener shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated load, and greater than or equal to the width divided by 16.

3. Transverse stiffeners shall extend a minimum of one-half the depth of the member except as required in Sections 510.10.5 and 510.10.7.

510.10.9 Additional Doubler Plate for Concentrated Forces

Doubler plates required for compression strength shall be designed in accordance with the requirements of Section 505.

Doubler plates required for tensile strength shall be designed in accordance with the requirements of Section 504.

Doubler plates required for shear strength (see Section 510.10.6) shall be designed in accordance with the provisions of Section 507.

In addition, doubler plates shall comply with the following criteria:

1. The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.
2. The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.

SECTION 511

DESIGN OF HSS AND BOX MEMBER CONNECTIONS

511.1 Concentrated Forces on HSS

511.1.1 Definitions of Parameters

- B = is the overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, mm
- B_p = is the width of plate, measured 90 degrees to the plane of the connection, mm
- D = is the outside diameter of round HSS member, mm
- F_y = is the specified minimum yield stress of HSS member material, MPa
- F_{yp} = is the specified minimum yield stress of plate, MPa
- F_u = is the specified minimum tensile strength of HSS material, MPa
- H = is the overall height of rectangular HSS member, measured in the plane of the connection, mm
- N = is the bearing length of the load, measured parallel to the axis of the HSS member, (or measured across the width of the HSS in the case of loaded cap plates), mm
- t = is the design wall thickness of HSS member, mm
- t_p = is the thickness of plate, mm

511.1.2 Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits of applicability:

1. Strength: $F_y \leq 360 \text{ MPa}$ for HSS
2. Ductility: $F_y/F_u \leq 0.8$ for HSS
3. Other limits apply for specific criteria

511.1.3 Concentrated Force Distributed Transversely

511.1.3.1 Criterion for Round HSS

When a concentrated force is distributed transversely to the axis of the HSS the design strength, ϕR_n , and the allowable strength, R_n/Ω , for the limit state of local yielding shall be determined as follows:

$$R_n = F_y t^2 [5.5 / (1 - 0.81 B_p / D)] Q_f \quad (511.1-1)$$

$$\phi = 0.90 (LRFD) \quad \Omega = 1.67 (ASD)$$

where Q_f is given by Eq. 511.2-1.

Additional limits of applicability are

1. $0.2 < B_p/D \leq 1.0$
2. $D/t \leq 50$ for T-connections and $D/t \leq 40$ for cross-connections

511.1.3.2 Criteria for Rectangular HSS

When a concentrated force is distributed transversely to the axis of the HSS the design strength, ϕR_n , and the allowable strength, R_n/Ω , shall be the lowest value according to the limit states of local yielding due to uneven load distribution, shear yielding (punching) and sidewall strength.

Additional limits of applicability are

1. $0.25 < B_p/B \leq 1.0$
2. B/t for the loaded HSS wall ≤ 35
 - a. For the limit state of local yielding due to uneven load distribution in the loaded plate,

$$R_n = [10 F_{yt} / (B/t)] B_p \leq F_{yp} t_p B_p \quad (511.1-2)$$

$$\phi = 0.95 (LRFD) \quad \Omega = 1.58 (ASD)$$

- b. For the limit state of shear yielding (punching),

$$R_n = 0.6 F_y t [2 t_p + 2 B_{ep}] \quad (511.1-3)$$

$$\phi = 0.95 (LRFD) \quad \Omega = 1.58 (ASD)$$

where

$$B_{ep} = 10 B_p / (B/t) \leq B_p$$

This limit state need not be checked when $B_p > (B - 2t)$, nor when $B_p < 0.85B$.

- c. For the limit state of sidewall under tension loading, the available strength shall be taken as the strength for sidewall local yielding. For the limit state of sidewall under compression loading, available strength shall be taken as the lowest value obtained according to the limit states of sidewall local yielding, sidewall local crippling and sidewall local buckling.

This limit state need not be checked unless the chord member and branch member (connecting element) have the same width ($\beta = 1.0$).

- c.1.1 For the limit state of sidewall local yielding,

$$R_n = 2F_y t [5k + N] \quad (511.1-4)$$

$$\phi = 1.00(LRFD) \quad \Omega = 1.50(ASD)$$

where

k = outside corner radius of the HSS, which is permitted to be taken as **1.5t** if unknown, mm

- c.1.2 For the limit state of sidewall local crippling, in T-connections,

$$R_n = 1.6t^2 [1 + 3N/(H - 3t)] (EF_y)^{0.5} Q_f \quad (511.1-5)$$

$$\phi = 0.75(LRFD) \quad \Omega = 2.00(ASD)$$

where Q_f is given by Eq. 511.2-10.

- c.1.3 For the limit state of sidewall local buckling in cross-connections,

$$R_n = [48t^3/(H - 3t)] (EF_y)^{0.5} Q_f \quad (511.1-6)$$

$$\phi = 0.90(LRFD) \quad \Omega = 1.67(ASD)$$

Where Q_f is given by Eq. 511.2-10.

The nonuniformity of load transfer along the line of weld, due to the flexibility of the HSS wall in a transverse plate-to-HSS connection, shall be considered in proportioning such welds. This requirement can be satisfied by limiting the total effective weld length, L_e , of groove and fillet welds to rectangular HSS as follows:

$$L_e 2[10/(B/t)] [(F_y t)/(F_{yp} t_p)] Bp \leq 2Bp \quad (511.1-7)$$

where

L_e = total effective weld length for welds on both sides of the transverse plate, mm

In lieu of Eq. 511.1-7, this requirement may be satisfied by other rational approaches.

User Note: An upper limit on weld size will be given by the weld that develops the available strength of the connected element.

511.1.4 Concentrated Force Distributed Longitudinally at the Center of the HSS Diameter or Width, and Acting Perpendicular to the HSS Axis

When a concentrated force is distributed longitudinally along the axis of the HSS at the center of the HSS diameter or width, and also acts perpendicular to the axis direction of the HSS (or has a component perpendicular to the axis direction of the HSS), the design strength, ϕR_n , and the allowable strength, R_n/Ω , perpendicular to the HSS axis shall be determined for the limit state of chord plastification as follows.

511.1.4.1 Criterion for Round HSS

An additional limit of applicability is: $D/t \leq 50$ for T-connections and $D/t \leq 40$ for cross-connections

$$R_n = 5.5F_y t^2 (1 + 0.25N/D) Q_f \quad (511.1-8)$$

$$\phi = 0.90(LRFD) \quad \Omega = 1.67(ASD)$$

where Q_f is given by Eq. 511.2-1.

511.1.4.2 Criterion for Rectangular HSS

An additional limit of applicability is: B/t for the loaded HSS wall ≤ 40

$$R_n = [F_y t^2 / (1 - t_p/B)] [2N/B + 4(1 - t_p/B)^{0.5} Q_f] \quad (511.1-9)$$

$$\phi = 1.00(LRFD) \quad \Omega = 1.50(ASD)$$

where

$$Q_f = (1 - U^2)^{0.5}$$

U is given by Eq. 511.2-12

511.1.5 Concentrated Force Distributed Longitudinally at the Center of the HSS Width, and Acting Parallel to the HSS Axis

When a concentrated force is distributed longitudinally along the axis of a rectangular HSS, and also acts parallel but eccentric to the axis direction of the member, the connection shall be verified as follows:

$$F_{yptp} \leq F_{ut} \quad (511.1-10)$$

User Note: This provision is primarily intended for shear tab connections. Equation 511.1-10 precludes shear yielding (punching) of the HSS wall by requiring the plate (shear tab) strength to be less than the HSS wall strength. For bracing connections to HSS columns, where a load is applied by a longitudinal plate at an angle to the HSS axis, the connection design will be governed by the force component perpendicular to the HSS axis (see Section 511.1.4b).

511.1.6 Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate.

When a concentrated force acts on the end of a capped HSS, and the force is in the direction of the HSS axis, the design strength, ϕR_n , and the allowable strength, R_n/Ω , shall be determined for the limit states of wall local yielding (due to tensile or compressive forces) and wall local crippling (due to compressive forces only), with consideration for shear lag, as follows.

User Note: The procedure below presumes that the concentrated force has a dispersion slope of 2.5:1 through

the cap plate (of thickness t_p) and disperses into the two HSS walls of dimension B .

If $(5t_p + N) \geq B$, the available strength of the HSS is computed by summing the contributions of all four HSS walls.

If $(5t_p + N) < B$, the available strength of the HSS is computed by summing the contributions of the two walls into which the load is distributed.

a. For the limit state of wall local yielding, for one wall,

$$R_n = F_y t [5t_p + N] \leq B F_y t \quad (511.1-11)$$

$$\phi = 1.00(LRFD) \quad \Omega = 1.50(ASD)$$

b. For the limit state of wall local crippling, for one wall,

$$R_n = 0.8t^2 [1 + (6N/B)(t/t_p)^{1.5}] [EF_y t_p/t]^{0.5} \quad (511.1-12)$$

$$\phi = 0.75(LRFD) \quad \Omega = 2.00(ASD)$$

511.2 HSS-to-HSS Truss Connections

HSS-to-HSS truss connections are defined as connections that consist of one or more branch members that are directly welded to a continuous chord that passes through the connection and shall be classified as follows:

1. When the punching load ($P_r \sin \theta$) in a branch member is equilibrated by beam shear in the chord member, the connection shall be classified as a T-connection when the branch is perpendicular to the chord and a Y-connection otherwise.
2. When the punching load ($P_r \sin \theta$) in a branch member is essentially equilibrated (within 20 percent) by loads in other branch member(s) on the same side of the connection, the connection shall be classified as a K-connection. The relevant gap is between the primary branch members whose loads equilibrate. An N-connection can be considered as a type of K-connection.

User Note: A K-connection with one branch perpendicular to the chord is often called an N-connection.

3. When the punching load ($P_r \sin \theta$) is transmitted through the chord member and is equilibrated by branch member(s) on the opposite side, the connection

shall be classified as a cross-connection.

4. When a connection has more than two primary branch members or branch members in more than one plane, the connection shall be classified as a general or multiplanar connection.

When branch members transmit part of their load as K-connections and part of their load as T-, Y-, or cross-connections, the nominal strength shall be determined by interpolation on the proportion of each in total.

For the purposes of this Specification, the centerlines of branch members and chord members shall lie in a common plane. Rectangular HSS connections are further limited to have all members oriented with walls parallel to the plane. For trusses that are made with HSS that are connected by welding branch members to chord members, eccentricities within the limits of applicability are permitted without consideration of the resulting moments for the design of the connection.

511.2.1 Definitions of Parameters

- B = overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, mm
- B_b = overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, mm
- D = outside diameter of round HSS main member, mm
- D_b = outside diameter of round HSS branch member, mm
- e = eccentricity in a truss connection, positive being away from the branches, mm
- F_y = specified minimum yield stress of HSS main member material, MPa
- F_{yb} = specified minimum yield stress of HSS branch member material, MPa
- F_u = specified minimum tensile strength of HSS material, MPa
- g = gap between toes of branch members in a gapped K-connection, neglecting the welds, mm
- H = overall height of rectangular HSS main member, measured in the plane of the connection, mm
- H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, mm
- t = design wall thickness of HSS main member, mm
- t_b = design wall thickness of HSS branch member, mm
- β = the width ratio; the ratio of branch diameter to chord diameter = D_b/D for round HSS; the ratio of overall branch width to chord width = B_b/B for rectangular HSS
- β_{eff} = the effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width

- γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness = $D/2t$ for round HSS; the ratio of one-half the width to wall thickness = $B/2t$ for rectangular HSS
- η = the load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N/B , where $N = H_b/\sin \theta$
- θ = acute angle between the branch and chord (degrees)
- ξ = the gap ratio; the ratio of the gap between the branches of a gapped K connection to the width of the chord = g/B for rectangular HSS

511.2.2 Criteria for Round HSS

The interaction of stress due to chord member forces and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f .

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \quad (511.2-1)$$

where U is the utilization ratio given by

$$U = |P_r/A_g F_c + M_r/SF_c| \quad (511.2-2)$$

and

- P_r = required axial strength in chord, N ; for K-connections, P_r is to be determined on the side of the joint that has the lower compression stress (lower U)
- M_r = required flexural strength in chord, N-mm
- A_g = chord gross area, mm²
- F_c = available stress, MPa
- S = chord elastic section modulus, mm³

For design according to Section 502.3.3 (LRFD):

- $P_r = P_u$ = required axial strength in chord, using LRFD load combinations, N
- $M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm
- $F_c = F_y$, MPa

For design according to Section 502.3.4 (ASD):

- $P_r = P_a$ = required axial strength in chord, using ASD load combinations, N
 $M_r = M_a$ = required flexural strength in chord, using ASD load combinations, $N\text{-mm}$
 $F_c = 0.6F_y$, MPa

511.2.2.1 Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits of applicability:

1. Joint eccentricity: $-0.55D \leq e \leq 0.25D$, where D is the chord diameter and e is positive away from the branches.
2. Branch angle: $\phi \geq 30^\circ$.
3. Chord wall slenderness ratio of diameter to wall thickness less than or equal to 50 for T-, Y- and K-connections; less than or equal to 40 for cross-connections.
4. Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50.
5. Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to $0.05 E/F_y$.
6. Width ratio: $0.2 < D_b/D \leq 1.0$ in general, and $0.4 \leq D_b/D \leq 1.0$ for gapped K-connections.
7. If a gap connection: g greater than or equal to the sum of the branch wall thicknesses.
8. If an overlap connection: $25\% \leq O_v \leq 100\%$, where $O_v = (q/p) \times 100\%$. p is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal diameter, the thicker) branch is a "thru member" connected directly to the chord.
9. Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch.
10. Strength: $F_y \leq 360$ MPa for chord and branches.
11. Ductility: $F_y/F_u \leq 0.8$

511.2.2.2 Branches with Axial Loads in T-, Y- and Cross-Connections

For T- and Y- connections, the design strength of the branch ϕP_n , or the allowable strength of the branch, P_n/Ω , shall be the lower value obtained according to the limit states of chord plastification and shear yielding (punching).

1. For the limit state of chord plastification in T- and Y-connections,

$$P_n \sin \theta = F_y t^2 [3.1 + 15.6\beta^2] \gamma^{0.2} Q_f \quad (511.2-3)$$

$$\phi = 0.90 (LRFD) \quad \Omega = 1.67 (ASD)$$

2. For the limit state of shear yielding (punching),

$$P_n = 0.6F_y \pi D_b [(1 + \sin \theta)/2 \sin^2 \theta] \quad (511.2-4)$$

$$\phi = 0.95 (LRFD) \quad \Omega = 1.58 (ASD)$$

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

3. For the limit state of chord plastification in cross-connections,

$$P_n \sin \theta = F_y t^2 [5.7/(1 - 0.81\beta)] Q_f \quad (511.2-5)$$

$$\phi = 0.90 (LRFD) \quad \Omega = 1.67 (ASD)$$

511.2.2.3 Branches with Axial Loads in K-Connections

For K-connections, the design strength of the branch, ϕP_n , and the allowable strength of the branch, P_n/Ω , shall be the lower value obtained according to the limit states of chord plastification for gapped and overlapped connections and shear yielding (punching) for gapped connections only.

1. For the limit state of chord plastification,

$$\phi = 0.90 (LRFD) \quad \Omega = 1.67 (ASD)$$

For the compression branch:

$$P_n \sin \theta = F_y t^2 [2.0 + 11.33 D_b/D] Q_g Q_f \quad (511.2-6)$$

where D_b refers to the compression branch only, and

$$Q_g = \gamma^{0.2} \left(1 + \frac{0.24\gamma^{1.2}}{e^{\left(\frac{0.5g}{t} - 1.33\right)} + 1} \right) \quad (511.2-7)$$

In gapped connections, g (measured along the crown of the chord neglecting weld dimensions) is positive. In overlapped connections, g is negative and equals q .

For the tension branch,

$$P_n \sin \theta = (P_n \sin \theta)_{\text{compression branch}} \quad (511.2-8)$$

2. For the limit state of shear yielding (punching) in gapped K-connections,

$$P_n = 0.6F_y t \pi D_b [(1 + \sin \theta) / 2 \sin^2 \theta] \quad (511.2-9)$$

$$\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$$

511.2.3 Criteria for Rectangular HSS

The interaction of stress due to chord member forces and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f .

1. When the chord is in tension,

$$Q_f = 1$$

2. When the chord is in compression in T-, Y-, and cross-connections,

$$Q_f = 1.3 - 0.4 U / \beta \leq 1 \quad (511.2-10)$$

3. When the chord is in compression in gapped K-connections,

$$Q_f = 1.3 - 0.4 U / \beta_{eff} \leq 1 \quad (511.2-11)$$

4. Where U is the utilization ratio given by

$$U = |P_r / A_g F_c + M_r / S F_c| \quad (511.2-12)$$

where

P_r = required axial strength in chord, N. For gapped K-connections, P_r is to be determined on the side of the joint that has the higher compression stress (higher U).

M_r = required flexural strength in chord, N-mm

A_g = chord gross area, mm²

F_c = available stress, MPa

S = chord elastic section modulus, mm³

For design according to Section 502.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using

LRFD load combinations, N

$M_r = M_u$ = required flexural strength in chord, using

LRFD load combinations, N-mm

$F_c = F_y$, MPa

For design according to Section 502.3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using

ASD load combinations, N

$M_r = M_u$ = required flexural strength in chord, using

ASD load combinations, N-mm

$F_c = 0.6F_y$, MPa

511.2.3.1 Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits:

1. Joint eccentricity: $-0.55H \leq e \leq 0.25H$, where H is the chord depth and e is positive away from the branches
2. Branch angle: $\theta \geq 30^\circ$
3. Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to 30 for overlapped K-connections
4. Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35
5. Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to $1.1(E/F_{yb})^{0.5}$ for overlapped K-connections
6. Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25 for T-, Y-, cross- and overlapped K-connections; greater than or equal to 0.35 for gapped K-connections

7. Aspect ratio: $0.5 \leq \text{ratio of depth to width} \leq 2.0$

8. Overlap: $25\% \leq O_v \leq 100\%$, where

$O_v = (q/p) \times 100\%$. p is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal width, the thicker) branch is a "thru member" connected directly to the chord

9. Branch width ratio for overlap connections: ratio of overall wall width of overlapping branch to overall wall width of overlapped branch greater than or equal to 0.75
10. Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch
11. Strength: $F_y \leq 360$ MPa for chord and branches
12. Ductility: $F_y/F_u \leq 0.8$
13. Other limits apply for specific criteria

511.2.3.2 Branches with Axial Loads in T-, Y- and Cross-Connections

For T-, Y-, and cross-connections, the design strength of the branch, ϕP_n , or the allowable strength of the branch, P_n/Ω , shall be the lowest value obtained according to the limit states of chord wall plastification, shear yielding (punching), sidewall strength and local yielding due to uneven load distribution. In addition to the limits of applicability in Section 511.2.3a, β shall not be less than 0.25.

1. For the limit state of chord wall plastification,

$$P_n \sin \theta = F_y t^2 [2\eta/(1 - \beta) + 4/(1 - \beta)^{0.5}] Q_f \quad (511.2-13)$$

$$\phi = 1.00 (LRFD) \quad \Omega = 1.50 (ASD)$$

This limit state need not be checked when $\beta > 0.85$.

2. For the limit state of shear yielding (punching),

$$P_n \sin \theta = 0.6 F_y t B [2\eta + 2\beta_{eop}] \quad (511.2-14)$$

$$\phi = 0.95 (LRFD) \quad \Omega = 1.58 (ASD)$$

In Eq. 511.2-14, the effective outside punching parameter $\beta_{eop} = 5\beta/\gamma$ shall not exceed β .

This limit state need not be checked when $\beta > (1 - 1/\gamma)$, nor when $\beta < 0.85$ and $B/t \geq 10$.

3. For the limit state of sidewall strength, the available strength for branches in tension shall be taken as the available strength for sidewall local yielding. For the limit state of sidewall strength, the available strength for branches in compression shall be taken as the lower of the strengths for sidewall local yielding and sidewall local crippling. For cross-connections with a branch angle less than 90 degrees, an additional check for chord sidewall shear failure must be made in accordance with Section 507.5.

This limit state need not be checked unless the chord member and branch member have the same width ($\beta = 1.0$).

- a. For the limit state of local yielding,

$$P_n \sin \theta = 2F_y t [5k + N] \quad (511.2-15)$$

$$\phi = 1.00 (LRFD) \quad \Omega = 1.50 (ASD)$$

where

k = outside corner radius of the HSS, which is permitted to be taken as **1.5t** if unknown, mm

N = bearing length of the load, parallel to the axis of the HSS main member
 $H_b/\sin \theta$, mm

- b. For the limit state of sidewall local crippling, in T- and Y-connections,

$$P_n \sin \theta = 1.6t^2 [1 + 3N/(H - 3t)] (EF_y)^{0.5} Q_f \quad (511.2-16)$$

$$\phi = 0.75 (LRFD) \quad \Omega = 2.00 (ASD)$$

- c. For the limit state of sidewall local crippling in cross-connections,

$$P_n \sin \theta = [48t^3/(H - 3t)] (EF_y)^{0.5} Q_f \quad (511.2-17)$$

$$\phi = 0.90 (LRFD) \quad \Omega = 1.67 (ASD)$$

- d. For the limit state of local yielding due to uneven load distribution,

$$P_n = F_{yb}t_b[2H_b + 2b_{eoi} - 4t_b] \quad (511.2-17)$$

$$\phi = 0.95 (LRFD) \quad \Omega = 1.58 (ASD)$$

where

$$b_{eoi} = [10/(B/t)][F_y t / F_{yb} t_b] B_b \leq B_b \quad (511.2-19)$$

This limit state need not be checked when $\beta < 0.85$.

511.2.3.3 Branches with Axial Loads in Gapped K – Connections

For gapped K-connections, the design strength of the branch, ϕP_n , or the allowable strength of the branch, P_n/Ω , shall be the lowest value obtained according to the limit states of chord wall plastification, shear yielding (punching), shear yielding and local yielding due to uneven load distribution. In addition to the limits of applicability in Section K2.3a, the following limits shall apply:

1. $B_b/B \geq 0.1 + \gamma/50$.
2. $\beta_{eff} \geq 0.35$.
3. $\zeta \geq 0.5(1 - \beta_{eff})$
4. Gap: g greater than or equal to the sum of the branch wall thicknesses.
5. The smaller $B_b > 0.63$ times the larger B_b .
 - a. For the limit state of chord wall plastification,

$$P_n \sin \theta = F_y t^2 [9.8 \beta_{eff} \gamma^{0.5}] Q_f \quad (511.2-20)$$

$$\phi = 0.90 (LRFD) \quad \Omega = 1.67 (ASD)$$

- b. For the limit state of shear yielding (punching),

$$P_n \sin \theta = 0.6 F_y t B [2\eta + \beta + \beta_{eop}] \quad (511.2-21)$$

$$\phi = 0.95 (LRFD) \quad \Omega = 1.58 (ASD)$$

In the above equation, the effective outside punching parameter $\beta_{eop} = 5\beta/\gamma$ shall not exceed β .

This limit state need only be checked if $B_b < (B - 2t)$ or the branch is not square.

- c. For the limit state of shear yielding of the chord in the gap, available strength shall be checked in accordance with Section 507.5. This limit state need only be checked if the chord is not square.
- d. For the limit state of local yielding due to uneven load distribution,

$$P_n = F_{yb}t_b[2H_b + B_b + b_{eoi} - 4t_b] \quad (511.2-22)$$

$$\phi = 0.95 (LRFD) \quad \Omega = 1.58 (ASD)$$

where

$$b_{eoi} = [10/(B/t)][F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (511.2-23)$$

This limit state need only be checked if the branch is not square or $B/t < 15$.

511.2.3.4 Branches with Axial Loads in Overlapped K – Connections

For overlapped K-connections, the design strength of the branch, ϕP_n , or the allowable strength of the branch, P_n/Ω , shall be determined from the limit state of local yielding due to uneven load distribution,

$$\phi = 0.95 (LRFD) \quad \Omega = 1.58 (ASD)$$

For the overlapping branch, and for overlap $25\% \leq O_v \leq 50\%$ measured with respect to the overlapping branch,

$$P_n = F_{yb} t_{bi} [(O_v/50)(2H_{bi} - 4t_{bi}) + b_{eoi} + b_{eov}] \quad (511.2-24)$$

For the overlapping branch, and for overlap **50% ≤ O_v < 80%** measured with respect to the overlapping branch,

$$P_n = F_{ybi} t_{bi} [2H_{bi} - 4t_{bi} + b_{eoi} + b_{eov}] \quad (511.2-25)$$

For the overlapping branch, and for overlap **80% ≤ O_v ≤ 100%** measured with respect to the overlapping branch,

$$P_n = F_{ybi} t_{bi} [2H_{bi} - 4t_{bi} + B_{bi} + b_{eov}] \quad (511.2-26)$$

where

b_{eoi} = is the effective width of the branch face welded to the chord,

$$b_{eoi} = [10/(B/t)] [(F_y t)/(F_{ybi} t_{bi})] B_{bi} \leq B_{bi} \quad (511.2-27)$$

b_{eov} = is the effective width of the branch face welded to the overlapped brace

$$= [10/(B_{bj}/t_{bj})] [(F_{ybj} t_{bj})/(F_{ybi} t_{bi})] B_{bi} \leq B_{bi} \quad (511.2-28)$$

B_{bi} = overall branch width of the overlapping branch, mm

B_{bj} = overall branch width of the overlapped branch, mm

F_{ybi} = specified minimum yield stress of the overlapping branch material, MPa

F_{ybj} = specified minimum yield stress of the overlapped branch material, MPa

H_{bi} = overall depth of the overlapping branch, mm

t_{bi} = thickness of the overlapping branch, mm

t_{bj} = thickness of the overlapped branch, mm

For the overlapped branch, P_n shall not exceed P_n of the overlapping branch, calculated using Eq. 511.2-24, 511.2-25, or 511.2-26, as applicable, multiplied by the factor $(A_{bj} F_{ybj}/A_{bi} F_{ybi})$,

where

A_{bi} = cross-sectional area of the overlapping branch

A_{bj} = cross-sectional area of the overlapped branch

511.2.3e Welds to Branches

The nonuniformity of load transfer along the line of weld, due to differences in relative flexibility of HSS walls in HSS-to-HSS connections, shall be considered in proportioning such welds. This can be considered by limiting the total effective weld length, L_e , of groove and fillet welds to rectangular HSS as follows:

1. In T-, Y- and cross-connections, for $\theta \leq 50$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + (B_b - 1.2t_b) \quad (511.2-29)$$

for $\theta \geq 60$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} \quad (511.2-30)$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

2. In gapped K-connections, around each branch, for $\theta \leq 50$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + 2(B_b - 1.2t_b) \quad (511.2-31)$$

for $\theta \geq 60$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + (B_b - 1.2t_b) \quad (511.2-32)$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees. In lieu of the above criteria in Eqs. 511.2-29 to 511.2-32, other rational criteria are permitted.

511.3 HSS-to-HSS Moment Connections

HSS-to-HSS moment connections are defined as connections that consist of one or two branch members that are directly welded to a continuous chord that passes through the connection, with the branch or branches loaded by bending moments.

A connection shall be classified

1. As a T-connection when there is one branch and it is perpendicular to the chord and as a Y-connection when there is one branch but not perpendicular to the chord.
2. As a cross-connection when there is a branch on each (opposite) side of the chord.

For the purposes of this Specification, the centerlines of the branch member(s) and the chord member shall lie in a common plane.

511.3.1 Definitions of Parameters

- B = overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, mm
- B_b = overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, mm
- D = outside diameter of round HSS main member, mm
- D_b = outside diameter of round HSS branch member, mm
- F_y = specified minimum yield stress of HSS main member, MPa
- F_{yb} = specified minimum yield stress of HSS branch member, MPa
- F_u = ultimate strength of HSS member, MPa
- H = overall height of rectangular HSS main member, measured in the plane of the connection, mm
- H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, mm
- t = design wall thickness of HSS main member, mm
- t_b = design wall thickness of HSS branch member, mm
- β = the width ratio; the ratio of branch diameter to chord diameter = D_b/D for round HSS; the ratio of overall branch width to chord width = B_b/B for rectangular HSS
- γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness = $D/2t$ for round HSS; the ratio of one-half the width to wall thickness = $B/2t$ for rectangular HSS

- η = the load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N/B , where $N = H_b/\sin \theta$
- θ = acute angle between the branch and chord (degrees)

511.3.2 Criteria for Round HSS

The interaction of stress due to chord member forces and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f .

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \quad (511.3-1)$$

where U is the utilization ratio given by,

$$U = |P_r/A_g F_c + M_r/SF_c| \quad (511.3-2)$$

where

- P_r = required axial strength in chord, N
- M_r = required flexural strength in chord, N-mm
- A_g = chord gross area, mm²
- F_c = available stress, MPa
- S = chord elastic section modulus, mm³

For design according to Section 502.3.3 (LRFD):

- $P_r = P_u$ = required axial strength in chord, using LRFD load combinations, N
- $M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm
- $A_g = F_y$, MPa

For design according to Section 502.3.4 (ASD):

- $P_r = P_a$ = required axial strength in chord, using ASD load combinations, N
- $M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm
- $F_c = 0.6F_y$, MPa

511.3.2.1 Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits of applicability:

1. Branch angle: $\theta \geq 30^\circ$.
2. Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for T- and Y-connections; less than or equal to 40 for cross-connections.
3. Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50.
4. Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to $0.05E/F_y$.
5. Width ratio: $2 < D_b/D \leq 1.0$.
6. Strength: $F_y \leq 360 \text{ MPa}$ for chord and branches.
7. Ductility: $F_y/F_u \leq 0.8$.

511.3.2.2 Branches with In-Plane Bending Moments in T-, Y- and Cross-Connections

The design strength, ϕM_n , and the allowable strength, M_n/Ω , shall be the lowest value obtained according to the limit states of chord plastification and shear yielding (punching).

1. For the limit state of chord plastification,

$$M_n \sin \theta = 5.39 F_y t^2 \gamma^{0.5} \beta D_b Q_f \quad (511.3-3)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

2. For the limit state of shear yielding (punching),

$$M_n = 0.6 F_y t D_b^2 [(1 + 3 \sin \theta) / 4 \sin^2 \theta] \quad (511.3-4)$$

$$\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$$

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

511.3.2.3 Branches with Out-of-Plane Bending Moments in T-, Y- and Cross-Connections

The design strength, ϕM_n , and the allowable strength, M_n/Ω , shall be the lowest value obtained according to the limit states of chord plastification and shear yielding (punching).

1. For the limit state of chord plastification,

$$M_n \sin \theta = F_y t^2 D_b [3.0 / (1 - 0.81\beta)] Q_f \quad (511.3-5)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

2. For the limit state of shear yielding (punching),

$$M_n = 0.6 F_y t D_b^2 [(3 + \sin \theta) / 4 \sin^2 \theta] Q_f \quad (511.3-6)$$

$$\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$$

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

511.3.2.4 Branches with Combined Bending Moment and Axial Force in T-, Y and Cross - Connections

Connections subject to branch axial load, branch in-plane bending moment, and branch out-of-plane bending moment, or any combination of these load effects, should satisfy the following.

For design according to Section 502.3.3 (LRFD):

$$(P_r / \phi P_n) + (M_{r-ip} / \phi M_{n-ip})^2 + (M_{r-op} / \phi M_{n-op})^2 \leq 1.0 \quad (511.3-7)$$

where

- P_r = P_u = required axial strength in branch, using LRFD load combinations, N
- ϕP_n = design strength obtained from Section 511.2.2b
- M_{r-ip} = required in-plane flexural strength in branch, using LRFD load combinations, N-mm
- ϕM_{n-ip} = design strength obtained from Section 511.3.2b
- M_{r-op} = required out-of-plane flexural strength in branch, using LRFD load combinations, N-mm
- ϕM_{n-op} = design strength obtained from Section 511.3.2c

For design according to Section 502.3.4 (ASD):

$$\frac{(P_r/(P_n/\Omega)) + (M_{r-ip}/(M_{n-ip}/\Omega))^2}{+ (M_{r-op}/(M_{n-op}/\Omega))} \leq 1.0 \quad (511.3-8)$$

where

- P_r = P_a = required axial strength in branch, using ASD load combinations, N
 P_n/Ω = allowable strength obtained from Section 511.2.2b
 M_{r-ip} = required in-plane flexural strength in branch, using ASD load combinations, N-mm
 M_{n-ip}/Ω = allowable strength obtained from Section 511.3.2b
 M_{r-op} = required out-of-plane flexural strength in branch, using ASD load combinations, N-mm
 M_{n-op}/Ω = allowable strength obtained from Section 511.3.2c

511.3.3 Criteria for Rectangular HSS

The interaction of stress due to chord member forces and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f .

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = (1.3 - 0.4U/\beta) \leq 1 \quad (511.3-9)$$

where U is the utilization ratio given by

$$U = |P_r/A_g F_c + M_r/SF_c| \quad (511.3-10)$$

where

- P_r = required axial strength in chord, N
 M_r = required flexural strength in chord, N-mm
 A_g = chord gross area, mm²
 F_c = available stress, MPa
 S = chord elastic section modulus, mm³

For design according to Section 502.3.3 (LRFD):

- $P_r = P_u$ = required axial strength in chord, using LRFD load combinations, N
 $M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm

$$F_c = F_y, \text{ MPa}$$

For design according to Section 502.3.4 (ASD):

- $P_r = P_a$ = required axial strength in chord, using ASD load combinations, N
 $M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm
 $F_c = 0.6F_y, \text{ MPa}$

511.3.3.1 Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits:

1. Branch angle is approximately 90°.
2. Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35.
3. Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35.
4. Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35.
5. Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25.
6. Aspect ratio: $0.5 \leq \text{ratio of depth to width} \leq 2.0$.
7. Strength: $F_y \leq 360 \text{ MPa}$ for chord and branches.
8. Ductility: $F_y/F_u \leq 0.8$.
9. Other limits apply for specific criteria

511.3.3.2 Branches with In-Plane Bending Moments in T- and Cross-Connections

The design strength, ϕM_n , and the allowable strength, M_n/Ω , shall be the lowest value obtained according to the limit states of chord wall plastification, sidewall local yielding and local yielding due to uneven load distribution.

1. For the limit state of chord wall plastification,

$$M_n = F_y t^2 H_b [(1/2\eta) + 2/(1-\beta)^{0.5} + \eta/(1-\beta)] Q_f \quad (511.3-11)$$

$$\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked when $\beta > 0.85$.

2. For the limit state of sidewall local yielding,

$$M_n = 0.5F_y^* t(H_b + 5t)^2 \quad (511.3-12)$$

$$\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

where

$$F_y^* = F_y \text{ for T-connections}$$

$$F_y^* = 0.8F_y \text{ for cross-connections}$$

This limit state need not be checked when $\beta < 0.85$.

3. For the limit state of local yielding due to uneven load distribution,

$$M_n = F_{yb}[Z_b - (1 - b_{eoi}/B_b)B_b H_b t_b] \quad (511.3-13)$$

$$\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$$

where

$$b_{eoi} = [10/(B/t)][F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (511.3-14)$$

Z_b = branch plastic section modulus about the axis of bending, mm³

This limit state need not be checked when $\beta < 0.85$.

511.3.3.3 Branches with Out-of-Plane Bending Moments in T- and Cross-Connections

The design strength, ϕM_n , and the allowable strength, M_n/Ω , shall be the lowest value obtained according to the limit states of chord wall plastification, sidewall local yielding, local yielding due to uneven load distribution and chord distortional failure.

1. For the limit state of chord wall plastification,

$$M_n = F_y t^2 [0.5H_b(1 + \beta)/(1 - \beta) + [2BB_b(1 + \beta)/(1 - \beta)]^{0.5}] Q_f \quad (511.3-15)$$

$$\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked when $\beta > 0.85$.

2. For the limit state of sidewall local yielding,

$$M_n = F_y^* t(B - t)(H_b + 5t) \quad (511.3-16)$$

$$\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

where

$$F_y^* = F_y \text{ for T-connections}$$

$$F_y^* = 0.8F_y \text{ for cross-connections}$$

This limit state need not be checked when $\beta < 0.85$.

3. For the limit state of local yielding due to uneven load distribution,

$$M_n = F_{yb}[Z_b - 0.51(1 - b_{eoi}/B_b)^2 B_b^2 t_b] \quad (511.3-17)$$

$$\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$$

where

$$b_{eoi} = [10/(B/t)][F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (511.3-18)$$

Z_b = branch plastic section modulus about the axis of bending, mm³

This limit state need not be checked when $\beta < 0.85$.

4. For the limit state of chord distortional failure,

$$M_n = 2F_y t [H_b t + [B H t (B + H)]^{0.5}] \quad (511.3-19)$$

$$\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked for cross-connections or for T-connections if chord distortional failure is prevented by other means.

511.3.3.4 Branches with Combined Bending Moment and Axial Force in T- and Cross-Connections

Connections subject to branch axial load, branch in-plane bending moment, and branch out-of-plane bending moment, or any combination of these load effects, should satisfy

For design according to Section 502.3.3 (LRFD)

$$\left(\frac{P_r}{\phi P_n} \right) + \left(\frac{M_{r-ip}}{\phi M_{n-ip}} \right) + \left(\frac{M_{r-op}}{\phi M_{n-op}} \right) \leq 1.0 \quad (511.3-20)$$

where

- P_r = P_u = required axial strength in branch, using LRFD load combinations, N
- ϕP_n = design strength obtained from Section 511.2.3b
- M_{r-ip} = required in-plane flexural strength in branch, using LRFD load combinations, N-mm
- ϕM_{n-ip} = design strength obtained from Section 511.3.3b
- M_{r-op} = required out-of-plane flexural strength in branch, using LRFD load combinations, N-mm
- ϕM_{n-op} = design strength obtained from Section 511.3.3c

For design according to Section 502.3.4 (ASD)

$$\left(\frac{P_r}{(P_n/\Omega)} \right) + \left(\frac{M_{r-ip}}{(M_{n-ip}/\Omega)} \right) + \left(\frac{M_{r-op}}{(M_{n-op}/\Omega)} \right) \leq 1.0 \quad (511.3-21)$$

where

- P_r = P_a = required axial strength in branch, using ASD load combinations, N
- P_n/Ω = allowable strength obtained from Section 511.2.3b
- M_{r-ip} = required in-plane flexural strength in branch, using ASD load combinations, N-mm
- M_{n-ip}/Ω = allowable strength obtained from Section 511.3.3b
- M_{r-op} = required out-of-plane flexural strength in branch, using ASD load combinations, N-mm
- M_{n-op}/Ω = allowable strength obtained from Section 511.3.3c

SECTION 512 DESIGN FOR SERVICEABILITY

512.1 General Provisions

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage. Limiting values of structural behavior for serviceability (for example, maximum deflections, accelerations) shall be chosen with due regard to the intended function of the structure. Serviceability shall be evaluated using appropriate load combinations for the serviceability limit states identified.

User Note: Additional information on serviceability limit states, service loads and appropriate load combinations for serviceability requirements can be found in ASCE 7, Appendix B and its Commentary. The performance requirements for serviceability in this Section are consistent with those requirements. Service loads, as stipulated herein, are those that act on the structure at an arbitrary point in time. That is, the appropriate load combinations are often less severe than those in ASCE 7, Section 2.4, where the LRFD load combinations are given.

512.2 Camber

Where camber is used to achieve proper position and location of the structure, the magnitude, direction and location of camber shall be specified in the structural drawings.

User Note: Camber recommendations are provided in the Code of Standard Practice for Steel Buildings and Bridges.

512.3 Deflections

Deflections in structural members and structural systems under appropriate service load combinations shall not impair the serviceability of the structure.

User Note: Conditions to be considered include levelness of floors, alignment of structural members, integrity of building finishes, and other factors that affect the normal usage and function of the structure. For composite members, the additional deflections due to the shrinkage and creep of the concrete should be considered.

512.4 Drift

Drift of a structure shall be evaluated under service loads to provide for serviceability of the structure, including the integrity of interior partitions and exterior cladding. Drift under strength load combinations shall not cause collision with adjacent structures or exceed the limiting values of such drifts that may be specified by this code.

512.5 Vibration

The effect of vibration on the comfort of the occupants and the function of the structure shall be considered. The sources of vibration to be considered include pedestrian loading, vibrating machinery and others identified for the structure.

512.6 Wind-Induced Motion

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered.

512.7 Expansion and Contraction

The effects of thermal expansion and contraction of a building shall be considered. Damage to building cladding can cause water penetration and may lead to corrosion.

512.8 Connection Slip

The effects of connection slip shall be included in the design where slip at bolted connections may cause deformations that impair the serviceability of the structure. Where appropriate, the connection shall be designed to preclude slip. For the design of slip-critical connections see Sections 510.3.8 and 510.3.9.

User Note: For more information on connection slip, refer to the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts.

SECTION 513

FABRICATION, ERECTION AND QUALITY CONTROL

Drawings

Shop drawings shall be prepared in advance of fabrication and give complete information necessary for the fabrication of the component parts of the structure, including the location, type and size of welds and bolts. Erection drawings shall be prepared in advance of erection and give information necessary for erection of the structure. Shop and erection drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted connections. Shop and erection drawings shall be made with due regard to speed and economy in fabrication and erection.

513.2 Fabrication

513.2.1 Cambering, Curving and Straightening

Local application of heat or mechanical means is permitted to be used to introduce or correct camber, curvature and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 593 °C for A514/A514M and A852/A852M steel nor 649 °C for other steels.

513.2.2 Thermal Cutting

Thermally cut edges shall meet the requirements of AWS D1.1, Sections 5.15.1.2, 5.15.4.3 and 5.15.4.4 with the exception that thermally cut free edges that will be subject to calculated static tensile stress shall be free of round-bottom gouges greater than 5 mm deep and sharp V-shaped notches. Gouges deeper than 5 mm and notches shall be removed by grinding or repaired by welding.

Reentrant corners, except reentrant corners of beam copes and weld access holes, shall meet the requirements of AWS D1.1, Section A5.16. If another specified contour is required it must be shown on the contract documents. Beam copes and weld access holes shall meet the geometrical requirements of Section 510.1.6. Beam copes and weld access holes in shapes that are to be galvanized shall be ground. For shapes with a flange thickness not exceeding 50 mm the roughness of thermally cut surfaces of copes shall be no greater than a surface roughness value of **50 μ m** as defined in ASME B46.1 Surface Texture (Surface Roughness, Waviness, and Lay). For beam copes and weld access holes in which the curved part of the access hole is thermally cut in ASTM A6/A6M hot rolled shapes with a flange thickness exceeding 50 mm and welded built-

up shapes with material thickness greater than 50 mm, a preheat temperature of not less than 66 °C shall be applied prior to thermal cutting. The thermally cut surface of access holes in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm and built-up shapes with a material thickness greater than 50 mm shall be ground and inspected for cracks using magnetic particle inspection in accordance with ASTM E709. Any crack is unacceptable regardless of size or location.

User Note: The AWS Surface Roughness Guide for Oxygen Cutting (AWS C4.1-77) sample 3 may be used as a guide for evaluating the surface roughness of copes in shapes with flanges not exceeding 50 mm thick.

513.2.3 Planing of Edges

Planing or finishing of sheared or thermally cut edges of plates or shapes is not required unless specifically called for in the contract documents or included in a stipulated edge preparation for welding.

513.2.4 Welded Construction

The technique of welding, the workmanship, appearance and quality of welds, and the methods used in correcting nonconforming work shall be in accordance with AWS D1.1 except as modified in Section 510.2.

513.2.5 Bolted Construction

Parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a drift pin in bolt holes during assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

Bolt holes shall comply with the provisions of the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, Section 3.3 except that thermally cut holes shall be permitted with a surface roughness profile not exceeding 25 μm as defined in ASME B46.1. Gouges shall not exceed a depth of 2 mm.

Fully inserted finger shims, with a total thickness of not more than 6 mm within a joint, are permitted in joints without changing the strength (based upon hole type) for the design of connections. The orientation of such shims is independent of the direction of application of the load. The use of high-strength bolts shall conform to the requirements of the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, except as modified in Section 510.3.513.2.6 Compression Joints

Compression joints that depend on contact bearing as part of the splice strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing, or other suitable means.

513.2.7 Dimensional Tolerances

Dimensional tolerances shall be in accordance with the AISC Code of Standard Practice for Steel Buildings and Bridges.

513.2.8 Finish of Column Bases

Column bases and base plates shall be finished in accordance with the following requirements:

1. Steel bearing plates 50 mm or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 50 mm but not over 100 mm in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section), to obtain a satisfactory contact bearing. Steel bearing plates over 100 mm in thickness shall be milled for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section).
2. Bottom surfaces of bearing plates and column bases that are grouted to ensure full bearing contact on foundations need not be milled.
3. Top surfaces of bearing plates need not be milled when complete-joint penetration groove welds are provided between the column and the bearing plate.

513.2.9 Holes for Anchor Rods

Holes for anchor rods shall be permitted to be thermally cut in accordance with the provisions of Section 513.2.2.

513.2.10 Drain Holes

When water can collect inside HSS or box members, either during construction or during service, the member shall be sealed, provided with a drain hole at the base, or protected by other suitable means.

513.2.11 Requirements for Galvanized Members

Members and parts to be galvanized shall be designed, detailed and fabricated to provide for flow and drainage of pickling fluids and zinc and to prevent pressure build-up in enclosed parts.

User Note: See The Design of Products to be Hot-Dip Galvanized after Fabrication, American Galvanizer's Association, and ASTM A123, A153, A384 and A780 for useful information on design and detailing of galvanized members.

513.3 Shop Painting

513.3.1 General Requirements

Shop painting and surface preparation shall be in accordance with the provisions of the AISC Code of Standard Practice for Steel Buildings and Bridges. Shop paint is not required unless specified by the contract documents.

513.3.2 Inaccessible Surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

513.3.3 Contact Surfaces

Paint is permitted in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, Section 3.2.2(b).

513.3.4 Finished Surfaces

Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

513.3.5 Surfaces Adjacent to Field Welds

Unless otherwise specified in the design documents, surfaces within 50 mm of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

513.4 Erection

513.4.1 Alignment of Column Bases

Column bases shall be set level and to correct elevation with full bearing on concrete or masonry.

513.4.2 Bracing

The frame of steel skeleton buildings shall be carried up true and plumb within the limits defined in the AISC Code of Standard Practice for Steel Buildings and Bridges. Temporary bracing shall be provided, in accordance with the requirements of the Code of Standard Practice for Steel Buildings and Bridges, wherever necessary to support the loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as required for safety.

513.4.3 Alignment

No permanent bolting or welding shall be performed until the adjacent affected portions of the structure have been properly aligned.

513.4.4 Fit of Column Compression Joints and Base Plates

Lack of contact bearing not exceeding a gap of 2 mm, regardless of the type of splice used (partial-joint-penetration groove welded or bolted), is permitted. If the gap exceeds 2 mm, but is less than 6 mm, and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with nontapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material.

513.4.5 Field Welding

Shop paint on surfaces adjacent to joints to be field welded shall be wire brushed if necessary to assure weld quality. Field welding of attachments to installed embedments in contact with concrete shall be done in such a manner as to avoid excessive thermal expansion of the embedment which could result in spalling or cracking of the concrete or excessive stress in the embedment anchors.

513.4.6 Field Painting

Responsibility for touch-up painting, cleaning and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the design documents.

513.4.7 Field Connections

As erection progresses, the structure shall be securely bolted or welded to support the dead, wind and erection loads.

513.5 Quality Control

The fabricator shall provide quality control procedures to the extent that the fabricator deems necessary to assure that the work is performed in accordance with this Specification. In addition to the fabricator's quality control procedures, material and workmanship at all times may be subject to inspection by qualified inspectors representing the purchaser. If such inspection by representatives of the purchaser will be required, it shall be so stated in the design documents.

513.5.1 Cooperation

As far as possible, the inspection by representatives of the purchaser shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The purchaser's inspector shall schedule this work for minimum interruption to the work of the fabricator.

513.5.2 Rejections

Material or workmanship not in conformance with the provisions of this Chapter may be rejected at any time during the progress of the work. The fabricator shall receive copies of all reports furnished to the purchaser by the inspection agency.

513.5.3 Inspection of Welding

The inspection of welding shall be performed in accordance with the provisions of AWS D1.1 except as modified in Section 510.2. When visual inspection is required to be performed by AWS certified welding inspectors, it shall be so specified in the design documents. When nondestructive testing is required, the process, extent and standards of acceptance shall be clearly defined in the design documents.

513.5.4 Inspection of Slip-Critical High-Strength Bolted Connections.

The inspection of slip-critical high-strength bolted connections shall be in accordance with the provisions of the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts.

513.5.5 Identification of Steel

The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material identification, visible at least through the "fit-up" operation, for the main structural elements of each shipping piece.

APPENDIX A

INELASTIC ANALYSIS AND DESIGN

A-1.1 General Provisions

Inelastic analysis is permitted for design according to the provisions of Section 502.3.3 (LRFD). Inelastic analysis is not permitted for design according to the provisions of Section 502.3.4 (ASD) except as provided in Section A-1.3.

A-1.2 Materials

Members undergoing plastic hinging shall have a specified minimum yield stress not exceeding 450 MPa.

A-1.3 Moment Redistribution

Beams and girders composed of compact sections as defined in Section 502.4 and satisfying the unbraced length requirements of Section A-1.7, including composite members, may be proportioned for nine-tenths of the negative moments at points of support, produced by the gravity loading computed by an elastic analysis, provided that the maximum positive moment is increased by one-tenth of the average negative moments. This reduction is not permitted for moments produced by loading on cantilevers and for design according to Sections A-1.4 through A-1.8 of this appendix.

If the negative moment is resisted by a column rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial force and flexure, provided that the axial force does not exceed $0.15\phi_c F_y A_g$ for LRFD or $0.15F_y A_g / \Omega_c$ for ASD,

where

A_g = gross area of member, mm^2

F_y = specified minimum yield stress of the compression flange, MPa

ϕ_c = resistance factor for compression = 0.90

Ω_c = safety factor for compression = 1.67

A-1.4 Local Buckling

Flanges and webs of members subject to plastic hinging in combined flexure and axial compression shall be compact with width-thickness ratios less than or equal to the limiting λ_p defined in Table 502.4.1 or as modified as follows:

1. For webs of doubly symmetric wide flange members and rectangular HSS in combined flexure and compression

- a. For $P_u/\phi_b P_y \leq 0.125$

$$h/t_w \leq 3.76 \sqrt{\frac{E}{F_y}} \left(1 - \frac{2.75 P_u}{\phi_b P_y} \right) \quad (\text{A-1-1})$$

- b. For $P_u/\phi_b P_y > 0.125$

$$h/t_w \leq 1.12 \sqrt{\frac{E}{F_y}} \left(2.33 - \frac{P_u}{\phi_b P_y} \right) \geq 1.49 \sqrt{\frac{E}{F_y}} \quad (\text{A-1-2})$$

where

E = modulus of elasticity of steel = 200,000 MPa

F_y = specified minimum yield stress of the type of steel being used, MPa

h = as defined in Section 502.4.2, mm

P_u = required axial strength in compression, N

P_y = member yield strength, N

t_w = web thickness, mm

ϕ_b = resistance factor for flexure = 0.90

2. For flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression, flange cover plates, and diaphragm plates between lines of fasteners or welds

$$b/t \leq 0.94 \sqrt{E/F_y} \quad (\text{A-1-3})$$

where

b = as defined in Section 502.4.2, mm

t = as defined in Section 502.4.2, mm

3. For circular hollow sections in flexure

$$D/t \leq 0.045 E/F_y \quad (\text{A-1-4})$$

where

D = outside diameter of round HSS member, mm

A-1.5 Stability and Second-Order Effects

Continuous beams not subjected to axial loads and that do not contribute to lateral stability of framed structures may be designed based on a first-order inelastic analysis or a plastic mechanism analysis. Braced frames and moment frames may be designed based on a first-order inelastic analysis or a plastic mechanism analysis provided that stability and second-order effects are taken into account.

Structures may be designed on the basis of a second-order inelastic analysis. For beam-columns, connections and connected members, the required strengths shall be determined from a second-order inelastic analysis, where equilibrium is satisfied on the deformed geometry, taking into account the change in stiffness due to yielding.

A-1.5a Braced Frames

In braced frames designed on the basis of inelastic analysis, braces shall be designed to remain elastic under the design loads. The required axial strength for columns and compression braces shall not exceed, $\phi_c = (0.85 F_y A_g)$.

where

$$\phi_c = 0.90(\text{LRFD})$$

A-1.5b Moment Frames

In moment frames designed on the basis of inelastic analysis, the required axial strength of columns shall not exceed $\phi_c = (0.75 F_y A_g)$.

where

$$\phi_c = 0.90(\text{LRFD})$$

A-1.6 Columns and Other Compression Members

In addition to the limits set in Sections A-1.5.a and A-1.5.b, the required axial strength of columns designed on the basis of inelastic analysis shall not exceed the design strength, $\phi_c P_n$, determined according to the provisions of Section 505.3.

Design by inelastic analysis is permitted if the column slenderness ratio, L/r , does not exceed $4.71 \sqrt{E/F_y}$.

where

- L = laterally unbraced length of a member, mm
 r = governing radius of gyration, mm

User Note: A well-proportioned member will not be expected to reach this limit.

A-1.7 Beams and Other Flexural Members

The required moment strength, M_u , of beams designed on the basis of inelastic analysis shall not exceed the design strength, ϕM_n , where

$$M_n = M_p = F_y Z < 1.6 F_y S \quad (\text{A-1-6})$$

$$\phi_c = 0.90(\text{LRFD})$$

Design by inelastic analysis is permitted for members that are compact as defined in Section 502.4 and as modified in Section A-1.4.

The laterally unbraced length, L_b , of the compression flange adjacent to plastic hinge locations shall not exceed L_{pd} , determined as follows.

1. For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange loaded in the plane of the web:

$$L_{pd} = \left[0.12 + 0.076 \left(\frac{M_1}{M_2} \right) \right] \left(\frac{E}{F_y} \right) r_y \quad (\text{A-1-7})$$

where

- M_1 = smaller moment at end of unbraced length of beam, N-mm
 M_2 = larger moment at end of unbraced length of beam, N-mm
 r_y = radius of gyration about minor axis, mm
 (M_1/M_2) is positive when moments cause reverse curvature and negative for single curvature.

2. For solid rectangular bars and symmetric box beams:

$$L_{pd} = \left[0.17 + 0.10 \left(\frac{M_1}{M_2} \right) \right] \left(\frac{E}{F_y} \right) r_y \geq 0.10 \left(\frac{E}{F_y} \right) r_y \quad (\text{A-1-8})$$

There is no limit on L_b for members with circular or square cross sections or for any beam bent about its minor axis.

A-1.8 Members under Combined Forces

When inelastic analysis is used for symmetric members subject to bending and axial force, the provisions in Section 508.1 apply. Inelastic analysis is not permitted for members subject to torsion and combined torsion, flexure, shear and/or axial force.

A-1.9 Connections

Connections adjacent to plastic hinging regions of connected members shall be designed with sufficient strength and ductility to sustain the forces and deformations imposed under the required loads.

APPENDIX A-2 DESIGN FOR PONDING

A-2.1 Simplified Design for Ponding

The roof system shall be considered stable for ponding and no further investigation is needed if both of the following two conditions are met:

$$C_p + 0.9C_s \leq 0.25 \quad (\text{A-2-1})$$

$$I_d \geq 3940S^4 \quad (\text{A-2-2})$$

where

$$C_p = \frac{504L_sL_p^4}{I_p}$$

$$C_s = \frac{504SL_s^4}{I_s}$$

L_p = column spacing in direction of girder (length of primary members), m

L_s = column spacing perpendicular to direction of girder (length of secondary members), m

S = spacing of secondary members, m

I_p = moment of inertia of primary members, mm^4

I_s = moment of inertia of secondary members, mm^4

I_d = moment of inertia of the steel deck supported on secondary members, mm^4 per m

C_p = flexibility constant for the supporting beam

C_s = Flexibility constant for one foot width of the roof deck ($S = 1.0$)

For trusses and steel joists, the moment of inertia I_s shall be decreased 15 percent when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

A-2.2 Improved Design for Ponding

The provisions given below are permitted to be used when a more exact determination of framing stiffness is needed than that given in Section A-2.1.

For primary members, the stress index shall be

$$U_p = \left(\frac{0.8F_y - f_o}{f_o} \right)_p \quad (\text{A-2-3})$$

For secondary members, the stress index shall be

$$U_s = \left(\frac{0.8F_y - f_o}{f_o} \right)_s \quad (\text{A-2-4})$$

where

f_o = stress due to the load combination ($D + R$)

D = nominal dead load

R = nominal load due to rainwater or snow, exclusive of the ponding contribution, MPa

For roof framing consisting of primary and secondary members, the combined stiffness shall be evaluated as follows: enter Figure A-2-1 at the level of the computed stress index U_p determined for the primary beam; move horizontally to the computed C_s value of the secondary beams and then downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

A similar procedure must be followed using Figure A-2-2.

For roof framing consisting of a series of equally spaced wall-bearing beams, the stiffness shall be evaluated as follows. The beams are considered as secondary members supported on an infinitely stiff primary member. For this case, enter Figure A-2-2 with the computed stress index U_s . The limiting value of C_s is determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

User Note: The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia per meter of width normal to its span to $3940l^4\text{mm}^4/\text{m}$.

For roof framing consisting of metal deck spanning between beams supported on columns, the stiffness shall be evaluated as follows. Employ Figure A-2-1 or A-2-2 using as C_s the flexibility constant for a 1 m width of the roof deck ($S = 1.0$).

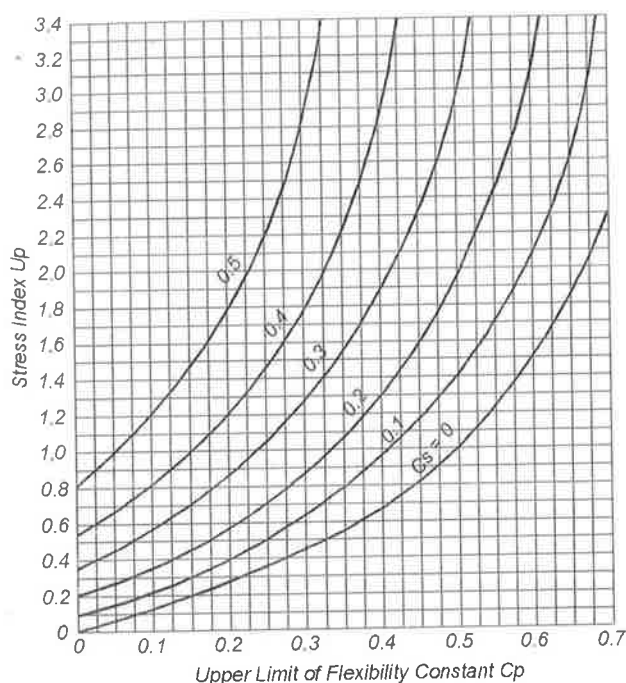


Figure A-2-1. Limiting flexibility coefficient for the primary systems.

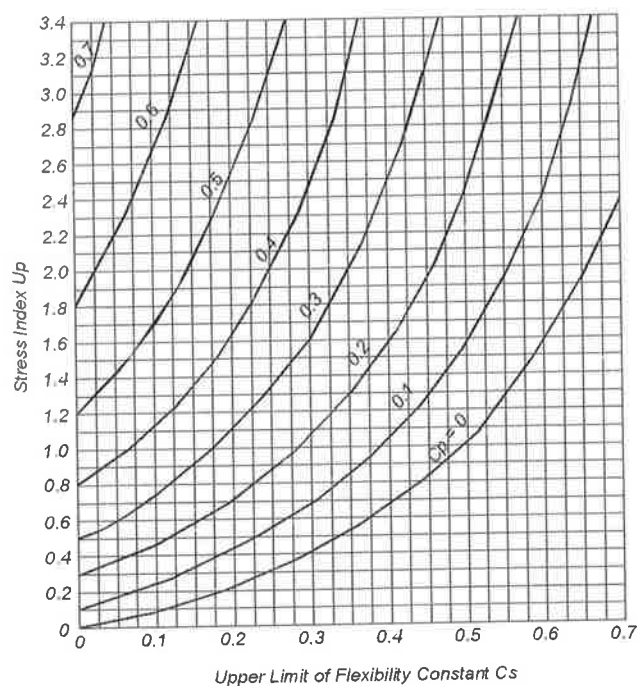


Figure A-2-2. Limiting flexibility coefficient for the secondary systems

APPENDIX A-3 DESIGN FOR FATIGUE

A-3.1 General Provisions

The provisions of this Appendix apply to stresses calculated on the basis of service loads. The maximum permitted stress due to unfactored loads is $0.66F_y$.

Stress range is defined as the magnitude of the change in stress due to the application or removal of the service live load. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation.

In the case of complete-joint-penetration groove welds, the maximum allowable design stress range calculated by Eq. A-3-1 applies only to welds with internal soundness meeting the acceptance requirements of Section 6.12.2 or 6.13.2 of AWS D1.1.

No evaluation of fatigue resistance is required if the live load stress range is less than the threshold stress range, F_{TH} . See Table A-3-1.

No evaluation of fatigue resistance is required if the number of cycles of application of live load is less than 20,000.

The cyclic load resistance determined by the provisions of this Appendix is applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions.

The cyclic load resistance determined by the provisions of this Appendix is applicable only to structures subject to temperatures not exceeding 300°F (150°C).

The engineer-of-record shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the connections.

A-3.2 Calculation of Maximum Stresses and Stress Ranges

Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of prying action, if any. In the case of axial stress combined with bending,

the maximum stresses, of each kind, shall be those determined for concurrent arrangements of the applied load.

For members having symmetric cross sections, the fasteners and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the stress range.

For axially loaded angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range.

A-3.3 Design Stress Range

The range of stress at service loads shall not exceed the design stress range computed as follows.

1. For stress categories A, B, B₋, C, D, E and E₋ the design stress range, F_{SR} , shall be determined by Eq. A-3-1.

$$F_{SR} = \left(\frac{C_f \times 329}{N} \right)^{0.333} \geq F_{TH} \quad (\text{A-3-1})$$

where

- F_{SR} = design stress range, MPa
 C_f = constant from Table A-3-1 for the category
 N = number of stress range fluctuations in design life
 = number of stress range fluctuations per day $\times 365 \times$ years of design life
 F_{TH} = threshold fatigue stress range, maximum stress range for indefinite design life from Table A-3-1, MPa

2. For stress category F, the design stress range, F_{SR} , shall be determined by Eq. A-3-2.

$$F_{SR} = \left(\frac{C_f \times 11 \times 10^4}{N} \right)^{0.167} \geq F_{TH} \quad (\text{A-3-2})$$

3. For tension-loaded plate elements connected at their end by cruciform, T, or corner details with complete-joint-penetration (CJP) groove welds or partial joint-penetration (PJP) groove welds, fillet welds, or combinations of the preceding, transverse to the direction of stress, the design stress range on the cross section of the tension-loaded plate element at the toe of the weld shall be determined as follows:

- a. Based upon crack initiation from the toe of the weld on the tension loaded plate element the design stress range, F_{SR} , shall be determined by Eq. A-3-3, for stress category C which is equal to

$$F_{SR} = \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \geq 68.9 \quad (\text{A-3-3})$$

- b. Based upon crack initiation from the root of the weld the design stress range, F_{SR} , on the tension loaded plate element using transverse PJP groove welds, with or without reinforcing or contouring fillet welds, the design stress range on the cross section at the toe of the weld shall be determined by Eq. A-3-4, stress category C' as follows:

$$F_{SR} = R_{PJP} \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \quad (\text{A-3-4})$$

where

R_{PJP} is the reduction factor for reinforced or nonreinforced transverse PJP groove welds determined as follows:

$$R_{PJP} = \left(\frac{1.12 - 1.01 \left(\frac{2a}{t_p} \right) + 1.24 \left(\frac{w}{t_p} \right)}{t_p^{0.167}} \right) \leq 1.0$$

If $R_{PJP} = 1.0$, use stress category C.

- $2a$ = the length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, mm
 w = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, mm
 t_p = thickness of tension loaded plate, mm

- c. Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range, F_{SR} , on the cross section at the toe of the welds shall be determined by Eq. A-3-5, stress category C as follows:

$$F_{SR} = R_{FIL} \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \quad (\text{A-3-5})$$

where

R_{FIL} is the reduction factor for joints using a pair of transverse fillet welds only.

$$R_{FIL} \left(\frac{0.10 + 1.24(w/t_p)}{t_p^{0.167}} \right) \leq 1.0$$

If $R_{FIL} = 1.0$, use stress category C.

A-3.4 Bolts and Threaded Parts

The range of stress at service loads shall not exceed the stress range computed as follows.

1. For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the design stress range computed using Eq. A-3-1 where C_f and F_{TH} are taken from Section 2 of Table A-3.1.
2. For high-strength bolts, common bolts, and threaded anchor rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial load and moment plus load due to prying action shall not exceed the design stress range computed using Eq. A-3-1.

The factor C_f shall be taken as 3.9×10^8 (as for stress category E'). The threshold stress, F_{TH} shall be taken as 48 MPa (as for stress category D). The net tensile area is given by Eq. A-3-6.

$$A_t = \frac{\pi}{4} (d_b - 0.9382P)^2 \quad (\text{A-3-6})$$

where

- P = pitch, mm per thread
 d_b = the nominal diameter (body or shank diameter), mm
 n = threads per mm

For joints in which the material within the grip is not limited to steel or joints which are not tensioned to the requirements of Table 510.3.1, all axial load and moment applied to the joint plus effects of any prying action shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the grip is limited to steel and which are tensioned to the requirements of Table 510.3.1, an analysis of the relative stiffness of the connected parts and bolts shall be permitted to be used to determine the tensile stress range in the pretensioned bolts due to the total service live load and moment plus effects of any prying action. Alternatively, the stress range in the bolts shall be assumed to be equal to the stress on the net tensile area due to 20 percent of the absolute value of the service load axial load and moment from dead, live and other loads.

A-3.5 Special Fabrication and Erection Requirements

Longitudinal backing bars are permitted to remain in place, and if used, shall be continuous. If splicing is necessary for long joints, the bar shall be joined with complete penetration butt joints and the reinforcement ground prior to assembly in the joint.

In transverse joints subject to tension, backing bars, if used, shall be removed and the joint back gouged and welded.

In transverse complete-joint-penetration T and corner joints, a reinforcing fillet weld, not less than 6 mm in size shall be added at re-entrant corners.

The surface roughness of flame cut edges subject to significant cyclic tensile stress ranges shall not exceed $25 \mu\text{m}$, where ASME B46.1 is the reference standard.

Reentrant corners at cuts, copes and weld access holes shall form a radius of not less than 10 mm by predrilling or subpunching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal surface.

For transverse butt joints in regions of high tensile stress, run-off tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Run-off tabs shall be removed and the end of the weld finished flush with the edge of the member.

See Section 510.2.2b for requirements for end returns on certain fillet welds subject to cyclic service loading.

Table A-3.1 Fatigue Design Parameters.

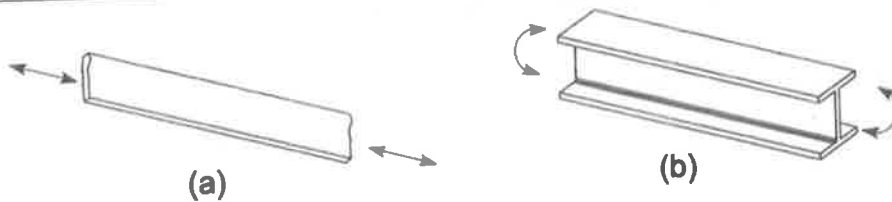
Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal, except non-coated weathering steel, with rolled or cleaned surface. Flame-cut edges with surface roughness value of 25 μm or less, but without reentrant corners.	<i>A</i>	250×10^8	165	Away from all welds or structural connections
1.2 Non-coated weathering steel base metal with rolled or cleaned surface. Flame-cut edges with surface roughness value of 25 μm or less, but without reentrant corners.	<i>B</i>	120×10^8	110	Away from all welds or structural connections
1.3 Member with drilled or reamed holes. Member with reentrant corners at copes, cuts, block-outs or other geometrical discontinuities made to requirements of Appendix 3.5, except weld access holes.	<i>B</i>	120×10^8	110	At any external edge or at hole perimeter
1.4 Rolled cross sections with weld access holes made to requirements of Section 510.1.6 and Appendix A-3.5. Members with drilled or reamed holes containing bolts for attachment of light bracing where there is a small longitudinal component of brace force.	<i>C</i>	44×10^8	69	At reentrant corner of weld access hole or at any small hole (may contain bolt for minor connections)
SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS				
2.1 Gross area of base metal in lap joints connected by high-strength bolts in joints satisfying all requirements for slip-critical connections.	<i>B</i>	120×10^8	110	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections.	<i>B</i>	120×10^8	110	In net section originating at side of hole
2.3 Base metal at the net section of other mechanically fastened joints except eye bars and pin plates	<i>D</i>	22×10^8	48	In net section originating at side of hole
2.4 Base metal at net section of eyebar head or pin plate.	<i>E</i>	11×10^8	31	In net section originating at side of hole

Table A-3.1 (cont.) Fatigue Design Parameters

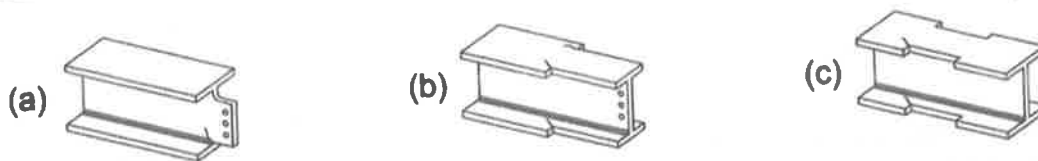
Illustrative Typical Examples

SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING

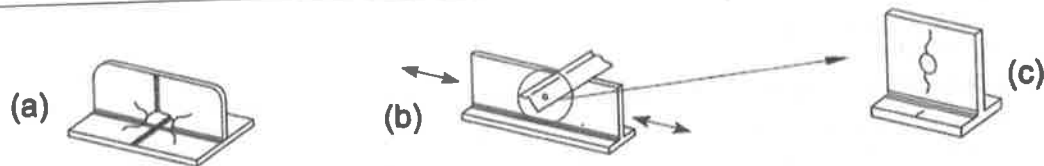
1.1 and 1.2



1.3

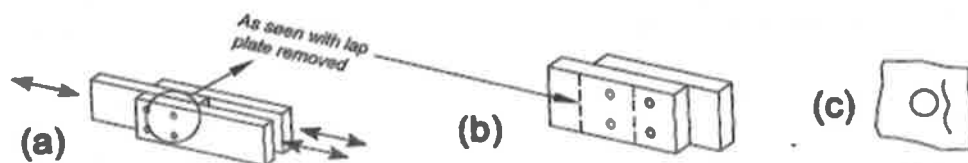


1.4

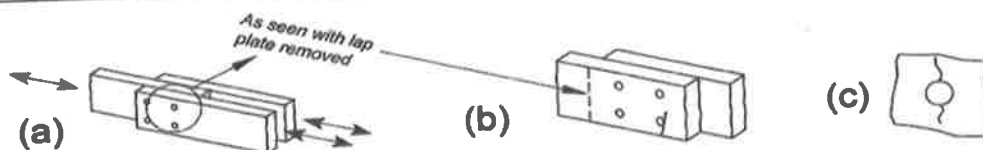


SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS

2.1



2.2



2.3



2.4

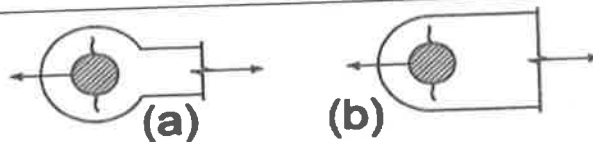


Table A-3.1 (cont.) Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete-joint-penetration groove welds, back gouged and welded from second side, or by continuous fillet welds.	B	120×10^8	110	From surface or internal discontinuities in weld away from end of weld
3.2 Base metal and weld metal in members without attachments built-up of plates or shapes, connected by continuous longitudinal complete-joint-penetration groove welds with backing bars not removed, or by continuous partial-joint-penetration groove welds.	B'	61×10^8	83	From surface or internal discontinuities in weld, including weld attaching backing bars
3.3 Base metal and weld metal termination of longitudinal welds at weld access holes in connected built-up members.	D	22×10^8	48	From the weld termination into the web or flange
3.4 Base metal at ends of longitudinal intermittent fillet weld segments.	E	11×10^8	31	In connected material at start and stop locations of any weld deposit
3.5 Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends of coverplates wider than the flange with welds across the ends. Flange thickness ≤ 20 mm	E	11×10^8	31	In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange with wide coverplates
Flange thickness > 20 mm	E'	3.9×10^8	18	
3.6 Base metal at ends of the partial length welded coverplates wider than the flange without welds across the ends.	E'	3.9×10^8	18	In edge of flange at end of coverplate weld
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections. Welds shall be on each side of the axis of the member to balance weld stresses.				Initiating from end of any weld termination extending into the base metal
$t \leq 20$ mm	E	11×10^8	31	
$t > 20$ mm	E'	3.9×10^8	18	

Table A-3.1 (cont.) Fatigue Design Parameters

Illustrative Typical Examples	
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS	
3.1	
3.2	
3.3	
3.4	
3.5	
3.6	
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS	
4.1	

Table A-3.1 (cont.) Fatigue Design Parameters

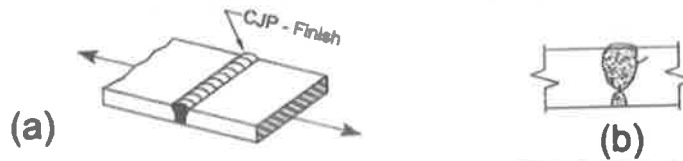
Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Base metal and weld metal in adjacent to complete-joint-penetration groove welded splices in rolled or welded cross sections with welds ground essentially parallel to the direction of stress.	B	120×10^8	110	From internal discontinuities in filler metal or along the fusion boundary
5.2 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in thickness or width made on a slope no greater than 8 to 20%				
$F_y < 620$ Mpa	B	120×10^8	110	From internal discontinuities in filler metal or along fusion boundary or at start of transition when $F_y \geq 620$ MPa
$F_y \geq 620$ Mpa	B'	61×10^8	83	
5.3 Base metal with F_y equal to or greater than 620 MPa and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius of not less than 600mm with the point of tangency at the end of the groove weld.	B	120×10^8	110	From internal discontinuities in filler metal or discontinuities along the fusion boundary
5.4 Base metal and weld metal in or adjacent to the toe of complete-joint-penetration T or corner joints or splices, with or without transitions in thickness having slopes no greater than 8 to 20%, when weld reinforcement is not removed.	C	44×10^8	69	From surface discontinuity at toe of weld extending into base metal or along fusion boundary.
5.5 Base metal and weld metal at transverse end connections of tension-loaded plate elements using partial-joint-penetration butt or T or corner joints, with reinforcing or contouring fillets, F_{SR} shall be the smaller of the toe crack or root crack stress range.				
Crack initiating from weld toe:	C	44×10^8	69	Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld
Crack initiating from weld root:	C'	Eqn. A-3-4	None provided	

Table A-3.1 (cont.) Fatigue Design Parameters

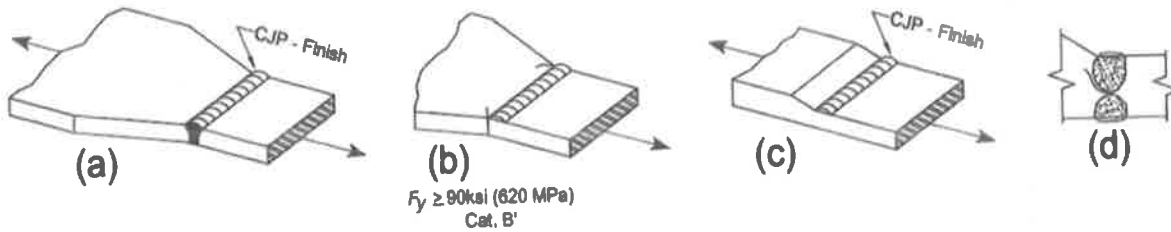
Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS

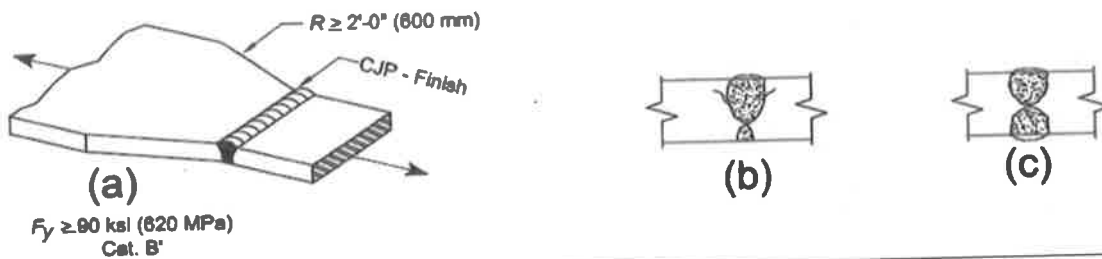
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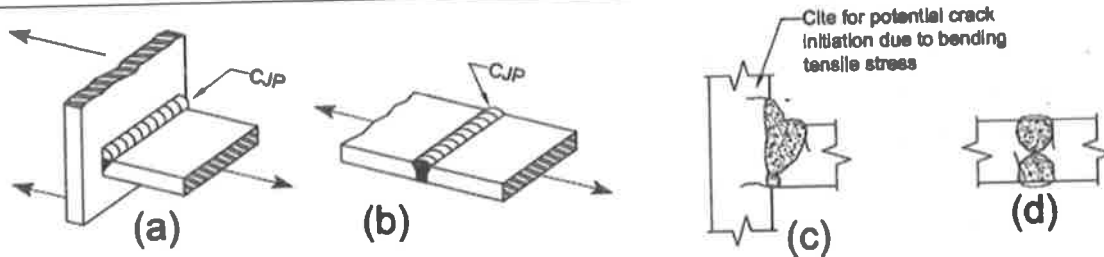
5.2



5.3



5.4



5.5

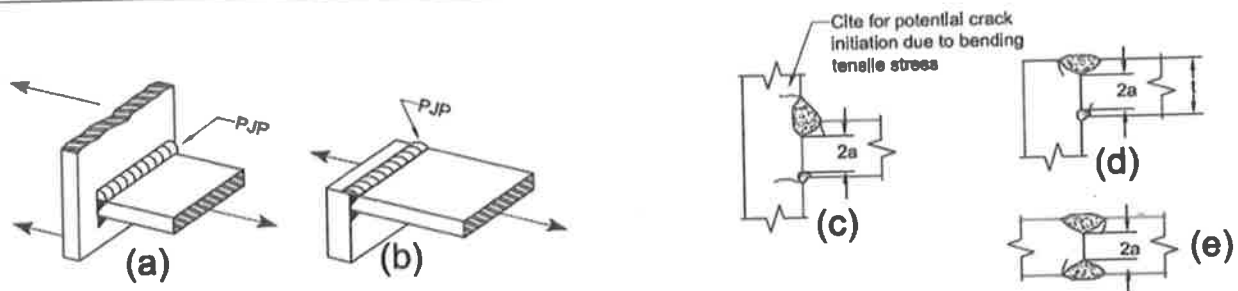


Table A-3.1 (cont.) Fatigue Design Parameters

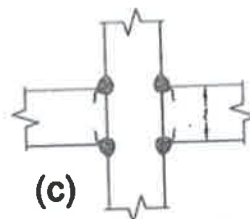
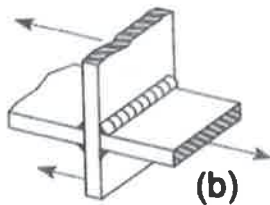
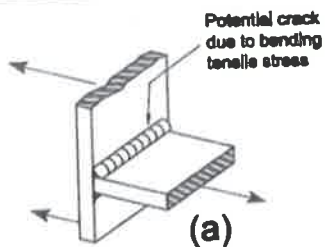
Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)				
5.6 Base metal and filler metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate F_{SR} shall be the smaller of the toe crack or root crack stress range.				
Crack initiating from weld toe:	C	44×10^8	69	Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld
Crack initiating from weld root:	C''	Eqn. A-3-5	None provided	
5.7 Base metal of tension-loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.	C	44×10^8	69	From geometrical discontinuity at toe of fillet extending into base metal
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.1 Base metal at details attached by complete-joint-penetration groove welds subject to longitudinal loading only when the detail embodies a transition radius R with the weld termination ground smooth.				
$R \geq 600$ mm	B	120×10^8	110	Near point of tangency of radius at edge of member
$600 \text{ mm} > R \geq 150$ mm	C	44×10^8	69	
$150 \text{ mm} > R \geq 50$ mm	D	22×10^8	48	
$50 \text{ mm} > R$	E	11×10^8	31	

Table A-3.1 (cont.) Fatigue Design Parameters

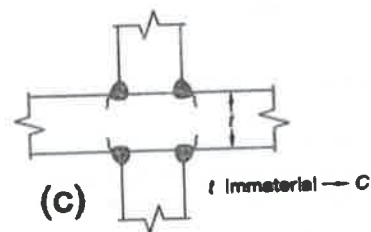
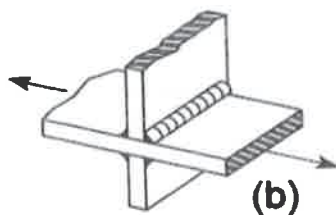
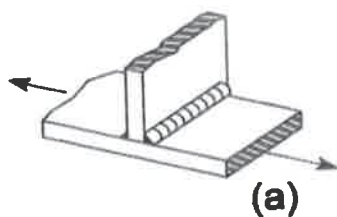
Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)

5.6



5.7



SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS

6.1

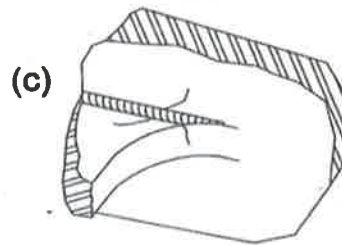
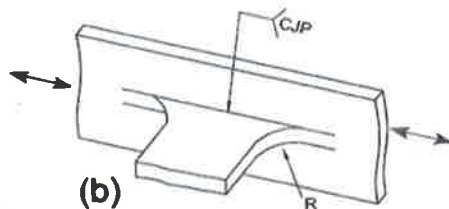
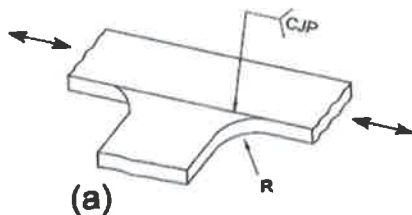
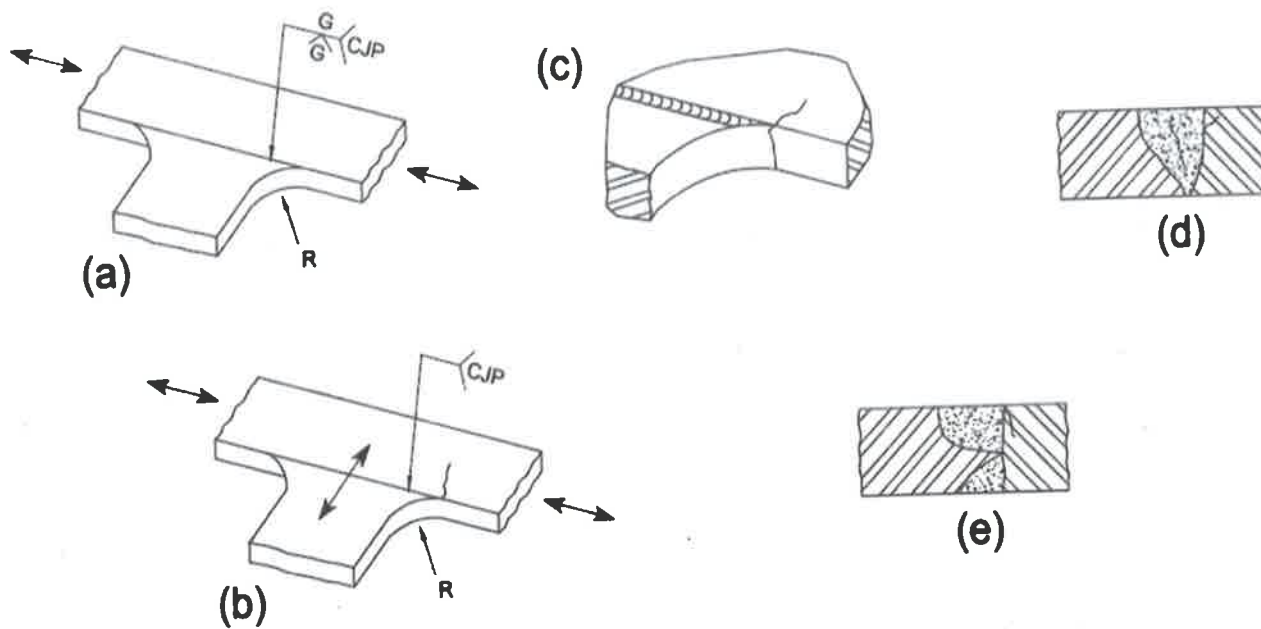


Table A-3.1 (cont.) Fatigue Design Parameters

Illustrative Typical Examples

SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.2



6.3

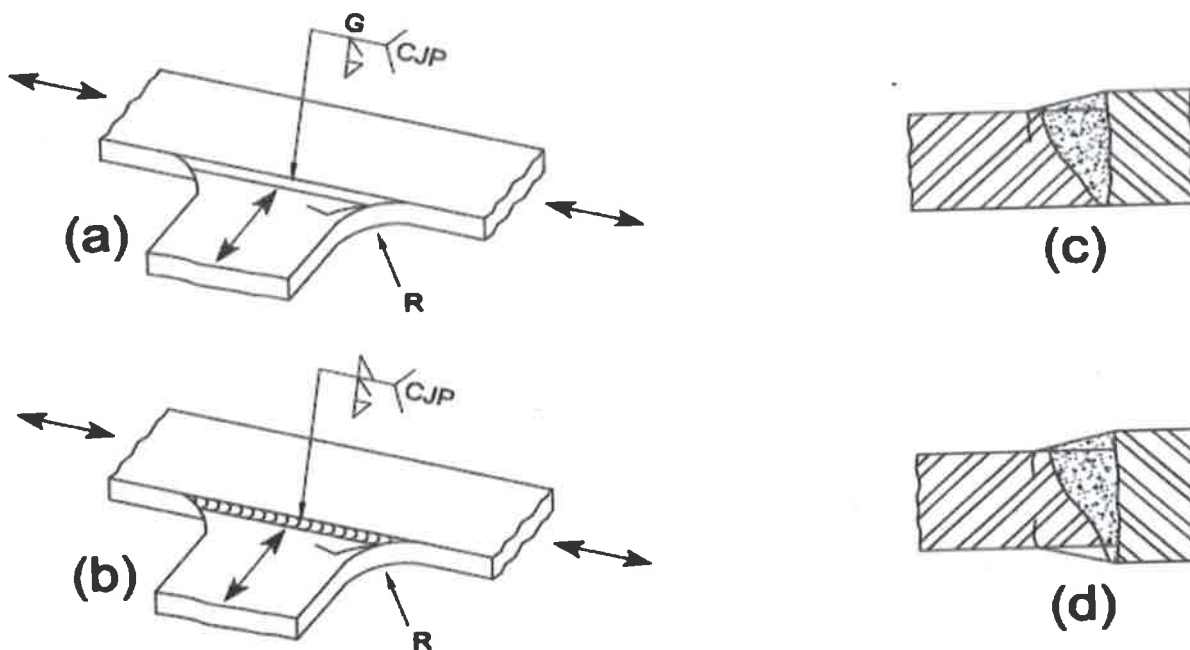


Table A-3.1 (cont.) Fatigue Design Parameters

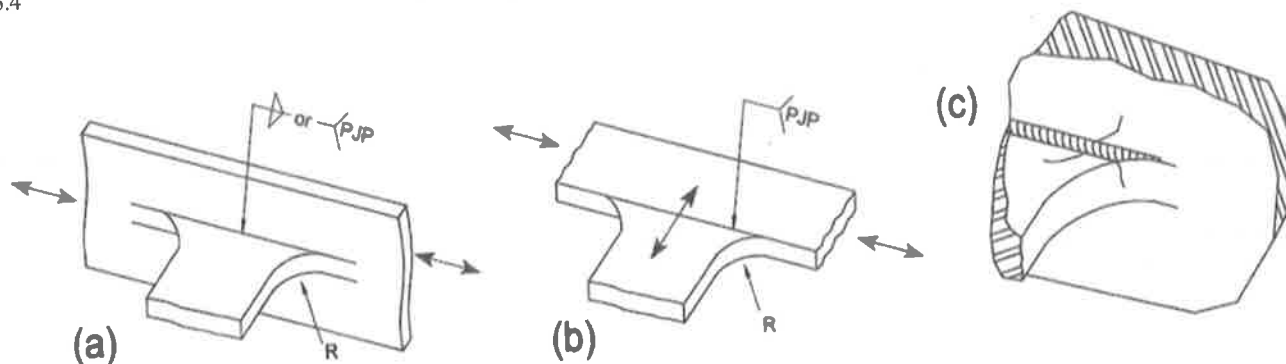
Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.4 Base metal subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius, R , with the weld termination ground smooth:				In weld termination or from the toe of the weld extending into member
$R > 50$ mm	D	22×10^8	48	
$R \leq 50$ mm	E	11×10^8	31	
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS¹				
7.1 Base metal subject to longitudinal loading at details attached by fillet welds parallel or transverse to the direction of stress where the detail embodies no transition radius and with detail length in direction of stress, a , and attachment height normal to the surface of the member, b :				
$a < 50$ mm	C	44×10^8	69	
$50 \text{ mm} \leq a \leq 12b$ or 100 mm	D	22×10^8	48	In the member at the end of the weld
$a > 12b$ or 100mm when b is ≤ 25 mm	E	11×10^8	31	
$a > 12b$ or 100mm when b is > 25 mm	E'	3.9×10^8	18	
7.2 Base metal subject to longitudinal stress at details attached by fillet or partial-joint-penetration groove welds, with or without transverse load on detail, when the detail embodies a transition, R , with weld termination ground smooth:				
$R > 50$ mm	D	22×10^8	48	In weld termination extending into member
$R \leq 50$ mm	E	11×10^8	31	
¹ “Attachments” as used herein, is defined as any steel detail welded to a member which, by its mere presence and independent of its loading, causes a discontinuity in the stress flow in the member and thus reduces the fatigue resistance.				

Table A-3.1 (cont.) Fatigue Design Parameters

Illustrative Typical Examples

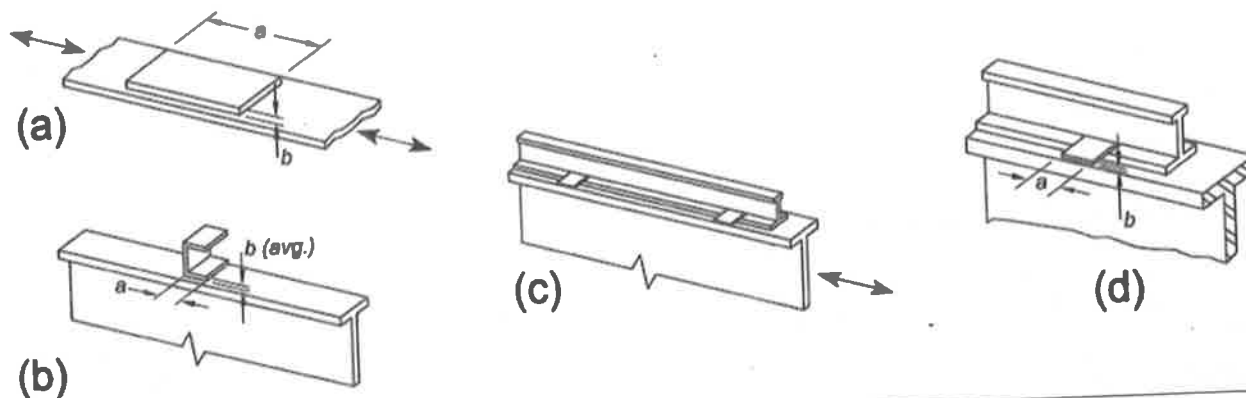
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.4



SECTION 7 – BASE METAL AT SHORT ATTACHMENTS

7.1



7.2

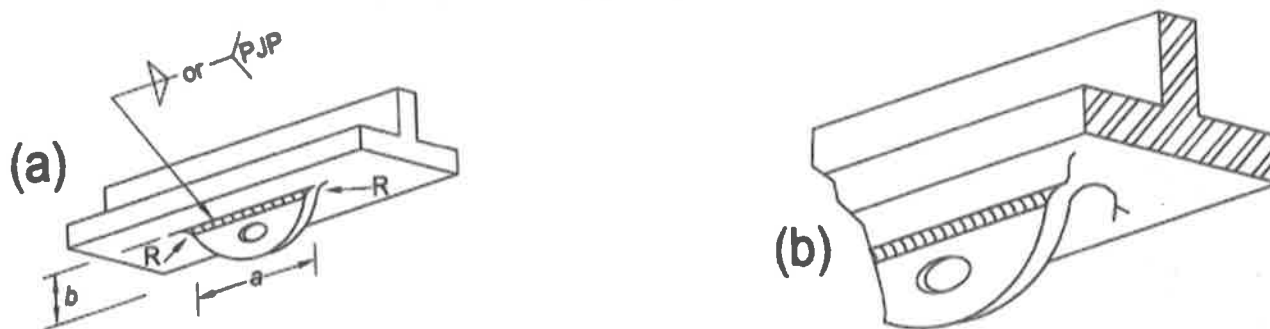


Table A-3.1 (cont.) Fatigue Design Parameters

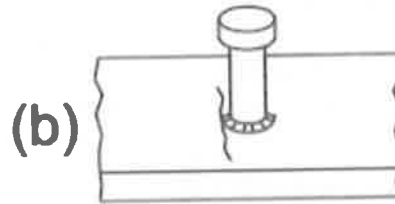
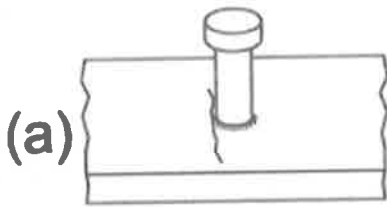
Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 8 - MISCELLANEOUS				
8.1 Base metal at stud-type shear connectors attached by fillet or electric stud welding.	C	44×10^8	69	At toe of weld in base metal
8.2 Shear on throat of continuous or intermittent longitudinal or transverse fillet welds.	F	150×10^{10} (Eq. A-3-2)	55	In throat of weld
8.3 Base metal at plug or slot welds.	E	11×10^8	31	At end of weld in base metal
8.4 Shear on plug or slot welds.	F	150×10^{10} (Eq. A-3-2)	55	At faying surface
8.5 Not fully tightened high-strength bolts, common bolts, threaded anchor rods and hanger rods with cut, ground or rolled threads. Stress range on tensile stress area due to live load plus prying action when applicable.	E'	3.9×10^8	48	At the root of the threads extending into the tensile stress area.

Table A-3.1 (cont.) Fatigue Design Parameters

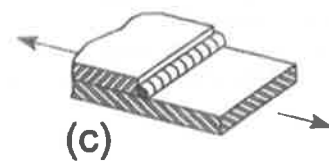
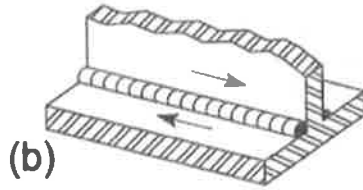
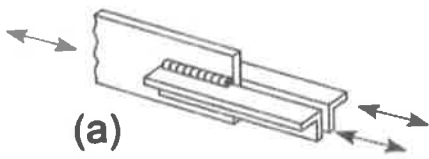
Illustrative Typical Examples

SECTION 8 - MISCELLANEOUS

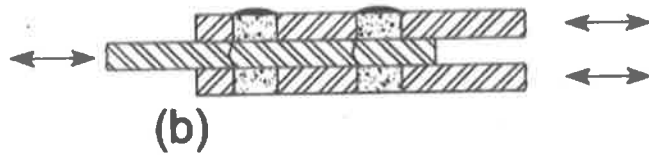
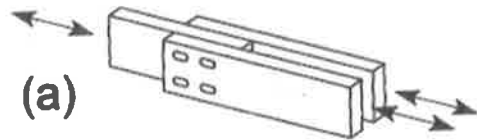
8.1



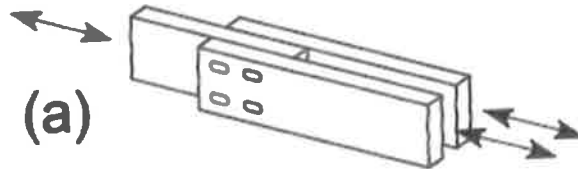
8.2



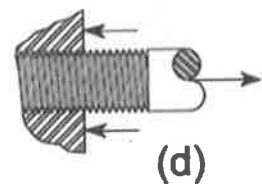
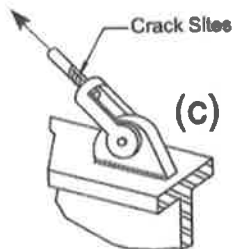
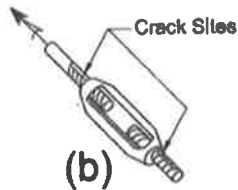
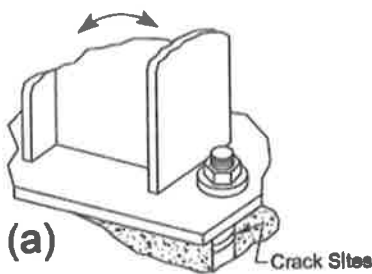
8.3



8.4



8.5



APPENDIX A-4 STRUCTURAL DESIGN FOR FIRE CONDITIONS

A-4.1 General Provisions

See Definitions.

A-4.1.1 Performance Objective

Structural components, members and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy.

Deformation criteria shall be applied where the means of providing structural fire resistance, or the design criteria for fire barriers, requires consideration of the deformation of the load-carrying structure.

Within the compartment of fire origin, forces and deformations from the design-basis fire shall not cause a breach of horizontal or vertical compartmentation.

A-4.1.2 Design by Engineering Analysis

The analysis methods in Section A-4.2 are permitted to be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Section A-4.2 provide evidence of compliance with performance objectives established in Section A-4.1.1.

The analysis methods in Section A-4.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the building code.

A-4.1.3 Design by Qualification Testing

The qualification testing methods in Section A-4.3 are permitted to be used to document the fire resistance of steel framing subject to the standardized fire testing protocols required by building codes.

A-4.1.4 Load Combinations and Required Strength

The required strength of the structure and its elements shall be determined from the following gravity load combination:

$$[0.9 \text{ or } 1.2]D + T + 0.5L + 0.2S \quad (\text{A-4-1})$$

where

D = nominal dead load

L = nominal occupancy live load

S = nominal snow load

T = nominal forces and deformations due to the design-basis fire defined in Section A-4.2.1

A lateral notional load, $N_i = 0.002Y_i$, as defined in Appendix A-7.2, where N_i = notional lateral load applied at framing level i and Y_i = gravity load from combination A-4-1 acting on framing level i , shall be applied in combination with the loads stipulated in Eq. A-4-1. Unless otherwise stipulated by the authority having jurisdiction, D , L and S shall be the nominal loads specified in ASCE 7.

A-4.2 Structural Design for Fire Conditions by Analysis

It is permitted to design structural members, components and building frames for elevated temperatures in accordance with the requirements of this section.

A-4.2.1 Design-Basis Fire

A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel load density based on the occupancy of the space shall be considered when determining the total fuel load. Heating conditions shall be specified either in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.

When the analysis methods in Section A-4.2 are used to demonstrate an equivalency as an alternative material or method as permitted by a building code, the design-basis fire shall be determined in accordance with ASTM E119.

Table A-4.2.1 Properties of Steel at Elevated Temperatures

Steel Temperature °C	$k_E = E_m/E$	$k_y = F_{ym}/F_y$	$k_u = F_{um}/F_y$
20	*	*	*
93	1.00	*	*
204	0.90	*	*
316	0.78	*	*
399	0.70	1.00	1.00
427	0.67	0.94	0.94
538	0.49	0.66	0.66
649	0.22	0.35	0.35
760	0.11	0.16	0.16
871	0.07	0.07	0.07
982	0.05	0.04	0.04
1093	0.02	0.02	0.02
1204	0.00	0.00	0.00
* Use ambient properties.			

A-4.2.1.1 Localized Fire

Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

A-4.2.1.2 Post-Flashover Compartment Fires

Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include fuel load, ventilation characteristics to the space (natural and mechanical), compartment dimensions and thermal characteristics of the compartment boundary.

A-4.2.1.3 Fire Duration

The fire duration in a particular area shall be determined by considering the total combustible mass, in other words, fuel load available in the space. In the case of either a localized fire or a post-flashover compartment fire, the time duration shall be determined as the total combustible mass divided by the mass loss rate, except where determined from Section A-4.2.1.2.

A-4.2.1.4 Exterior Fires

The exposure of exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be considered along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method identified in Section A-4.2.1.2 shall be used for describing the characteristics of the interior compartment fire.

A-4.2.1.5 Active Fire Protection Systems

The effects of active fire protection systems shall be considered when describing the design-basis fire.

Where automatic smoke and heat vents are installed in non-sprinklered spaces, the resulting smoke temperature shall be determined from calculation.

A-4.2.2 Temperatures in Structural Systems under Fire Conditions

Temperatures within structural members, components and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

Table A-4.2.2 Properties of Concrete at Elevated Temperatures

Concrete Temperature [°C]	$k_c = f'_{cm}/f'_c$		E_{cm}/E_c	$\epsilon_{cu}(\%)$
	NWC	LWC		LWC
20	1.00	1.00	1.00	0.25
93	0.95	1.00	0.93	0.34
204	0.90	1.00	0.75	0.46
288	0.86	1.00	0.61	0.58
316	0.83	0.98	0.57	0.62
427	0.71	0.85	0.38	0.80
538	0.54	0.71	0.20	1.06
649	0.38	0.58	0.092	1.32
760	0.21	0.45	0.073	1.43
871	0.10	0.31	0.055	1.49
982	0.05	0.18	0.036	1.50
1093	0.01	0.05	0.018	1.50
1204	0.00	0.00	0.00	-

For lightweight concrete (LWC), values of ϵ_{cu} shall be obtained from tests.

A-4.2.3 Material Strengths at Elevated Temperatures

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, it is permitted to use the material properties stipulated in this section. These relationships do not apply for steels with a yield strength in excess of 448MPa or concretes with specified compression strength in excess of 55 MPa.

A-4.2.3.1 Thermal Elongation

Thermal expansion of structural and reinforcing steels: For calculations at temperatures above 65°C, the coefficient of thermal expansion shall be $1.4 \times 10^{-5}/^\circ\text{C}$.

Thermal expansion of normal weight concrete: For calculations at temperatures above 65°C, the coefficient of thermal expansion shall be $1.8 \times 10^{-5}/^\circ\text{C}$.

Thermal expansion of lightweight concrete: For calculations at temperatures above 65°C, the coefficient of thermal expansion shall be $7.9 \times 10^{-6}/^\circ\text{C}$.

A-4.2.3.2 Mechanical Properties at Elevated Temperatures

The deterioration in strength and stiffness of structural members, components, and systems shall be taken into account in the structural analysis of the frame.

The values F_{ym} , F_{um} , E_m , f'_{cm} , E_{cm} , and ϵ_{cu} at elevated temperature to be used in structural analysis, expressed as the ratio with respect to the property at ambient, assumed to be 20°C, shall be defined as in Tables A-4.2.1 and A-4.2.2. It is permitted to interpolate between these values.

A-4.2.4 Structural Design Requirements

A-4.2.4.1 General Structural Integrity

The structural frame shall be capable of providing adequate strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The structural system shall be designed to sustain local damage with the structural system as a whole remaining stable.

Continuous load paths shall be provided to transfer all forces from the exposed region to the final point of resistance. The foundation shall be designed to resist the forces and to accommodate the deformations developed during the design-basis fire.

A-4.2.4.2 Strength Requirements and Deformation Limits

Conformance of the structural system to these requirements shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

Individual members shall be provided with adequate strength to resist the shears, axial forces and moments determined in accordance with these provisions.

Connections shall develop the strength of the connected members or the forces indicated above. Where the means of providing fire resistance requires the consideration of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

A-4.2.4.3 Methods of Analysis

A-4.2.4.3a Advanced Methods of Analysis

The methods of analysis in this section are permitted for the design of all steel building structures for fire conditions. The design-basis fire exposure shall be that determined in Section A-4.2.1. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The thermal response shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature dependent thermal properties of the structural elements and fire-resistive materials as per Section A-4.2.2.

The mechanical response results in forces and deflections in the structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicitly the deterioration in strength and stiffness with increasing temperature, the effects of thermal expansions and large deformations. Boundary conditions and connection fixity must represent the proposed structural design. Material properties shall be defined as per Section A-4.2.3.

The resulting analysis shall consider all relevant limit states, such as excessive deflections, connection fractures, and overall or local buckling.

A-4.2.4.3b Simple Methods of Analysis

The methods of analysis in this section are applicable for the evaluation of the performance of individual members at elevated temperatures during exposure to fire.

The support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.

1. Tension members

It is permitted to model the thermal response of a tension element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section A-4.2.1.

The design strength of a tension member shall be determined using the provisions of Section 504, with steel properties as stipulated in Section A-4.2.3 and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

2. Compression members

It is permitted to model the thermal response of a compression element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section A-4.2.1.

The design strength of a compression member shall be determined using the provisions of Section 505 with steel properties as stipulated in Section A-4.2.3.

3. Flexural members

It is permitted to model the thermal response of flexural elements using a one-dimensional heat transfer equation to calculate bottom flange temperature and to assume that this bottom flange temperature is constant over the depth of the member.

The design strength of a flexural member shall be determined using the provisions of Section 506 with steel properties as stipulated in Section A-4.2.3.

4. Composite floor members

It is permitted to model the thermal response of flexural elements supporting a concrete slab using a one-dimensional heat transfer equation to calculate bottom flange temperature. That temperature shall be taken as constant between the bottom flange and mid-depth of the web and shall decrease linearly by no more than 25 percent from the mid-depth of the web to the top flange of the beam.

The design strength of a composite flexural member shall be determined using the provisions of Section 509, with reduced yield stresses in the steel consistent with the temperature variation described under thermal response.

A-4.2.4.4 Design Strength

The design strength shall be determined as in Section 502.3.3. The nominal strength, R_n , shall be calculated using material properties, as stipulated in Section A-4.2.3, at the temperature developed by the design-basis fire.

A-4.3 Design by Qualification Testing**A-4.3.1 Qualification Standards**

Structural members and components in steel buildings shall be qualified for the rating period in conformance with ASTM E119. It shall be permitted to demonstrate compliance with these requirements using the procedures specified for steel construction in Section 5 of ASCE/SFPE 29.

A-4.3.2 Restrained Construction

For floor and roof assemblies and individual beams in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting actions caused by thermal expansion throughout the range of anticipated elevated temperatures.

Steel beams, girders and frames supporting concrete slabs that are welded or bolted to integral framing members (in other words, columns, girders) shall be considered restrained construction.

A-4.3.3 Unrestrained Construction

Steel beams, girders and frames that do not support a concrete slab shall be considered unrestrained unless the members are bolted or welded to surrounding construction that has been specifically designed and detailed to resist actions caused by thermal expansion.

A steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist effects of thermal expansion.

APPENDIX A-5 EVALUATION OF EXISTING STRUCTURES

A-5.1 General Provisions

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the available strength of a load resisting member or system. The evaluation shall be performed by structural analysis (Section A-5.3), by load tests (Section A-5.4), or by a combination of structural analysis and load tests, as specified in the contract documents. Where load tests are used, the engineer-of-record shall first analyze the structure, prepare a testing plan, and develop a written procedure to prevent excessive permanent deformation or catastrophic collapse during testing.

A-5.2 Material Properties

A-5.2.1 Determination of Required Tests

The Engineer-of-Record shall determine the specific tests that are required from Section A-5.2.2 through A-5.2.6 and specify the locations where they are required. Where available, the use of applicable project records shall be permitted to reduce or eliminate the need for testing.

A-5.2.2 Tensile Properties

Tensile properties of members shall be considered in evaluation by structural analysis (Section A-5.3) or load tests (Section A-5.4). Such properties shall include the yield stress, tensile strength and percent elongation. Where available, certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M or A568/A568M, as applicable, shall be permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure.

A-5.2.3 Chemical Composition

Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification (WPS). Where available, results from certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures shall be permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties, or from samples taken from the same locations.

A-5.2.4 Base Metal Notch Toughness

Where welded tension splices in heavy shapes and plates as defined in Section 501.3.1d are critical to the performance of the structure, the Charpy V-Notch toughness shall be determined in accordance with the provisions of Section 501.3.1d. If the notch toughness so determined does not meet the provisions of Section 501.3.1d, the engineer-of-record shall determine if remedial actions are required.

A-5.2.5 Weld Metal

Where structural performance is dependent on existing welded connections, representative samples of weld metal shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of AWS D1.1 are not met, the engineer-of-record shall determine if remedial actions are required.

A-5.2.6 Bolts and Rivets

Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine tensile strength in accordance with ASTM F606 or ASTM F606M and the bolt classified accordingly. Alternatively, the assumption that the bolts are ASTM A307 shall be permitted. Rivets shall be assumed to be ASTM A502, Grade 1, unless a higher grade is established through documentation or testing.

A-5.3 Evaluation by Structural Analysis

A-5.3.1 Dimensional Data

All dimensions used in the evaluation, such as spans, column heights, member spacings, bracing locations, cross section dimensions, thicknesses and connection details, shall be determined from a field survey. Alternatively, when available, it shall be permitted to determine such dimensions from applicable project design or shop drawings with field verification of critical values.

A-5.3.2 Strength Evaluation

Forces (load effects) in members and connections shall be determined by structural analysis applicable to the type of structure evaluated. The load effects shall be determined for the loads and factored load combinations stipulated in Section 502.2. The available strength of members and connections shall be determined from applicable provisions of Sections 502 through 511 of this Specification.

A-5.3.3 Serviceability Evaluation

Where required, the deformations at service loads shall be calculated and reported.

A-5.4 Evaluation by Load Tests**A-5.4.1 Determination of Load Rating by Testing**

To determine the load rating of an existing floor or roof structure by testing, a test load shall be applied incrementally in accordance with the engineer of record's plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

The tested strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested strength equal to $1.2D + 1.6L$, where D is the nominal dead load and L is the nominal live load rating for the structure.

The nominal live load rating of the floor structure shall not exceed that which can be calculated using applicable provisions of the specification. For roof structures, L_r , S , or R as defined in the Symbols, shall be substituted for L . More severe load combinations shall be used where required by this code. Periodic unloading shall be considered once the service load level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test load for one hour that the deformation of the structure does not increase by more than 10 percent above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay, representative of the most critical conditions, shall be selected.

A-5.4.2 Serviceability Evaluation

When load tests are prescribed, the structure shall be loaded incrementally to the service load level. Deformations shall be monitored for a period of one hour. The structure shall then be unloaded and the deformation recorded.

A-5.5 Evaluation Report

After the evaluation of an existing structure has been completed, the engineer-of-record shall prepare a report documenting the evaluation. The report shall indicate whether the evaluation was performed by structural analysis, by load testing or by a combination of structural analysis and load testing. Furthermore, when testing is performed, the report shall include the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design drawings, mill test reports and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the structure, including all members and connections, is adequate to withstand the load effects.

APPENDIX A-6 STABILITY BRACING FOR COLUMNS AND BEAMS

A-6.1 General Provisions

Bracing is assumed to be perpendicular to the members to be braced; for inclined or diagonal bracing, the brace strength (force or moment) and stiffness (force per unit displacement or moment per unit rotation) shall be adjusted for the angle of inclination. The evaluation of the stiffness furnished by a brace shall include its member and geometric properties, as well as the effects of connections and anchoring details.

Two general types of bracing systems are considered, relative and nodal. A relative brace controls the movement of the brace point with respect to adjacent braced points. A nodal brace controls the movement at the braced point without direct interaction with adjacent braced points. The available strength and stiffness of the bracing shall equal or exceed the required limits unless analysis indicates that smaller values are justified by analysis.

A second-order analysis that includes an initial out-of-straightness of the member to obtain brace strength and stiffness is permitted in lieu of the requirements of this appendix.

A-6.2 Column Bracing

It is permitted to brace an individual column at end and intermediate points along its length by either relative or nodal bracing systems. It is assumed that nodal braces are equally spaced along the column.

A-6.2.1 Relative Bracing

The required brace strength is

$$P_{br} = 0.004P_r \quad (\text{A-6-1})$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{2P_r}{L_b} \right) (\text{LRFD}) \quad \beta_{br} = \Omega \left(\frac{2P_r}{L_b} \right) (\text{ASD}) \quad (\text{A-6-2})$$

where

$$\phi = 0.75 (\text{LRFD}) \quad \Omega = 2.00 (\text{ASD})$$

L_b = distance between braces, mm

For design according to Section 502.3.3 (LRFD)

P_r = required axial compressive strength using LRFD load combinations, N

For design according to Section 502.3.4 (ASD)

P_r = required axial compressive strength using ASD load combinations, N

A-6.3.2 Nodal Bracing

The required brace strength is

$$P_{br} = 0.01P_r \quad (\text{A-6-3})$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{8P_r}{L_b} \right) (\text{LRFD}) \quad \beta_{br} = \Omega \left(\frac{8P_r}{L_b} \right) (\text{ASD}) \quad (\text{A-6-4})$$

$$\phi = 0.75 (\text{LRFD}) \quad \Omega = 2.00 (\text{ASD})$$

For design according to Section 502.3.3 (LRFD)

P_r = required axial compressive strength using LRFD load combinations, N

For design according to Section 502.3.4 (ASD)

P_r = required axial compressive strength using ASD load combinations, N

A-6.3 Beams Bracing

At points of support for beams, girders and trusses, restraint against rotation about their longitudinal axis shall be provided. Beam bracing shall prevent the relative displacement of the top and bottom flanges, in other words, twist of the section. Lateral stability of beams shall be provided by lateral bracing, torsional bracing or a combination of the two. In members subjected to double curvature bending, the inflection point shall not be considered a brace point.

A-6.3.1 Lateral Bracing

Bracing shall be attached near the compression flange, except for a cantilevered member, where an end brace shall be attached near the top (tension) flange. Lateral bracing shall be attached to both flanges at the brace point nearest the inflection point for beams subjected to double curvature bending along the length to be braced.

A-6.3.1.1.1 Relative Bracing

The required brace strength is

$$P_{br} = 0.008M_r C_d / h_o \quad (A-6-5)$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{4M_r C_d}{L_b h_o} \right) (LRFD) \quad \beta_{br} = \Omega \left(\frac{4M_r C_d}{L_b h_o} \right) (ASD) \quad (A-6-6)$$

where

$$\phi = 0.75 (LRFD) \quad \Omega = 2.00 (ASD)$$

h_o = distance between flange centroids, mm

C_d = 1.0 for bending in single curvature; 2.0 for double curvature; $C_d = 2.0$ only applies to the brace closest to the inflection point

L_b = laterally unbraced length, mm

For design according to Section 502.3.3 (LRFD)

M_r = required flexural strength using LRFD load combinations, N-mm

For design according to Section 502.3.4 (ASD)

M_r = required flexural strength using ASD load combinations, N-mm

A-6.3.1.1.2 Nodal Bracing

The required brace strength is

$$P_{br} = 0.02M_r C_d / h_o \quad (A-6-7)$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{10M_r C_d}{L_b h_o} \right) (LRFD) \quad \beta_{br} = \Omega \left(\frac{10M_r C_d}{L_b h_o} \right) (ASD) \quad (A-6-8)$$

where

$$\phi = 0.75 (LRFD) \quad \Omega = 2.00 (ASD)$$

For design according to Section 502.3.3 (LRFD)

M_r = required flexural strength using LRFD load combinations, N-mm

For design according to Section 502.3.4 (ASD)

M_r = required flexural strength using ASD load combinations, N-mm

When L_b is less than L_q , the maximum unbraced length for M_r , then L_b in

Eq. A-6-8 shall be permitted to be taken equal to L_q .

A-6.3.2 Torsional Bracing

It is permitted to provide either nodal or continuous torsional bracing along the beam length. It is permitted to attach the bracing at any cross-sectional location and it need not be attached near the compression flange. The connection between a torsional brace and the beam shall be able to support the required moment given below.

A-6.3.2.2.1 Nodal Bracing

The required bracing moment is

$$M_r = \frac{0.024M_r L_r}{nC_b L_b} \quad (A-6-9)$$

The required cross-frame or diaphragm bracing stiffness is

$$\beta_{TB} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{sec}}\right)} \quad (A-6-10)$$

where

$$\beta_T = \frac{1}{\phi} \left(\frac{2.4LM_r^2}{nEI_y C_b^2} \right) (LRFD) \quad \beta_T = \Omega \left(\frac{2.4LM_r^2}{nEI_y C_b^2} \right) (ASD) \quad (A-6-11)$$

$$\beta_{sec} = \frac{3.3E}{h_o} \left(\frac{1.5h_o t_w^3}{12} + \frac{t_s b_s^3}{12} \right) \quad (A-6-12)$$

where

$$\phi = 0.75 (LRFD) \quad \Omega = 3.00 (ASD)$$

User Note: $\Omega = 1.52/\phi = 3.00$ in Eq. A-6-11 because the moment term is squared.

- L = span length, mm
 n = number of nodal braced points within the span
 E = modulus of elasticity of steel = 200,000 MPa
 I_y = out-of-plane moment of inertia, mm⁴
 C_b = modification factor defined in Section 506
 t_w = beam web thickness, mm
 t_s = web stiffener thickness, mm
 b_s = stiffener width for one-sided stiffeners (use twice the individual stiffener width for pairs of stiffeners), mm
 β_T = brace stiffness excluding web distortion, N-mm/radian
 β_{sec} = web distortional stiffness, including the effect of web transverse stiffeners, if any, N-mm/radian

For design according to Section 502.3.3 (LRFD)

M_r = required flexural strength using LRFD load combinations, N-mm

For design according to Section 502.3.4 (ASD)

M_r = required flexural strength using ASD load combinations, N-mm

If $\beta_{sec} < \beta_T$, Eq. A-6-10 is negative, which indicates that torsional beam bracing will not be effective due to inadequate web distortional stiffness.

When required, the web stiffener shall extend the full depth of the braced member and shall be attached to the flange if the torsional brace is also attached to the flange. Alternatively, it shall be permissible to stop the stiffener short by a distance equal to $4t_w$ from any beam flange that is not directly attached to the torsional brace. When L_b is less than L_q , then L_b in Eq. A-6-9 shall be permitted to be taken equal to L_q .

A-6.3.1.2.2 Continuous Torsional Bracing

For continuous bracing, use Eqs. A-6-9, A-6-10 and A-6-13 with L/n taken as 1.0 and L_b taken as L_q ; the bracing moment and stiffness are given per unit span length. The distortional stiffness for an unstiffened web is

$$\beta_{sec} = \frac{3.3Et_w^3}{12h_o} \quad (\text{A-6-13})$$

A-6.4 Beam-Column Bracing

For bracing of beam-columns, the required strength and stiffness for the axial force shall be determined as specified in Section 6.2, and the required strength and stiffness for the flexure shall be determined as specified in Section 6.3. The values so determined shall be combined as follows:

- When relative lateral bracing is used, the required strength shall be taken as the sum of the values determined using Eq. A-6-1 and A-6-5, and the required stiffness shall be taken as the sum of the values determined using Eqs. A-6-2 and A-6-6.
- When nodal lateral bracing is used, the required strength shall be taken as the sum of the values determined using Eqs. A-6-3 and A-6-7, and the required stiffness shall be taken as the sum of the values determined using Eqs. A-6-4 and A-6-8. In Eqs. A-6-4 and A-6-8, L_b for beam-columns shall be taken as the actual unbraced length; the provisions in Section 6.2.2 and 6.3.1b that L_b need not be taken less than the maximum permitted effective length based upon P_r and M_r shall not be applied.
- When torsional bracing is provided for flexure in combination with relative or nodal bracing for axial force, the required strength and stiffness shall be combined or distributed in a manner that is consistent with the resistance provided by the element(s) of the actual bracing details.

APPENDIX A-7

DIRECT ANALYSIS METHOD

A-7.1 General Requirements

Members shall satisfy the provisions of Section 508.1 with the nominal column strengths, P_n , determined using $K = 1.0$. The required strengths for members, connections and other structural elements shall be determined using a second order elastic analysis with the constraints presented in Section A-7.3.

All component and connection deformations that contribute to the lateral displacement of the structure shall be considered in the analysis.

A-7.2 Notional Loads

Notional loads shall be applied to the lateral framing system to account for the effects of geometric imperfections, inelasticity, or both. Notional loads are lateral loads that are applied at each framing level and specified in terms of the gravity loads applied at that level. The gravity load used to determine the notional load shall be equal to or greater than the gravity load associated with the load combination being evaluated. Notional loads shall be applied in the direction that adds to the destabilizing effects under the specified load combination.

A-7.3 Design-Analysis Constraints

1. The second-order analysis shall consider both $P - \delta$ and $P - \Delta$ effects. It is permitted to perform the analysis using any general second-order analysis method, or by the amplified first-order analysis method of Section 503.2, provided that the B_1 and B_2 factors are based on the reduced stiffnesses defined in Eqs. A-7-2 and A-7-3. Analyses shall be conducted according to the design and loading requirements specified in either Section 502.3.3 (LRFD) or Section 502.3.4 (ASD). For ASD, the second-order analysis shall be carried out under 1.6 times the ASD load combinations and the results shall be divided by 1.6 to obtain the required strengths.

Methods of analysis that neglect the effects of $P - \delta$ on the lateral displacement of the structure are permitted where the axial loads in all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure satisfy the following limit:

$$\alpha P_r < 0.15 P_{eL} \quad (\text{A-7-1})$$

where

P_r = required axial compressive strength under LRFD or ASD load combinations, N

$P_{eL} = \pi^2 EI / L^2$, evaluated in the plane of bending

and $\alpha = 1.0(\text{LRFD}) \quad \alpha = 1.6(\text{ASD})$

2. A notional load, $N_i = 0.002 Y_i$, applied independently in two orthogonal directions, shall be applied as a lateral load in all load combinations. This load shall be in addition to other lateral loads, if any,

where

N_i = notional lateral load applied at level i , N

Y_i = gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i , N

The notional load coefficient of 0.002 is based on an assumed initial story out-of-plumbness ratio of 1/500. Where a smaller assumed out-of-plumbness is justified, the notional load coefficient may be adjusted proportionally.

For frames where the ratio of second-order drift to first-order drift is equal to or less than 1.5, it is permissible to apply the notional load, N_i , as a minimum lateral load for the gravity-only load combinations and not in combination with other lateral loads.

For all cases, it is permissible to use the assumed out-of-plumbness geometry in the analysis of the structure in lieu of applying a notional load or a minimum lateral load as defined above.

User Note: The unreduced stiffnesses (EI and AE) are used in the above calculations. The ratio of second-order drift to first-order drift can be represented by B_2 , as calculated using Eq. 503.2-3. Alternatively, the ratio can be calculated by comparing the results of a second-order analysis to the results of a first-order analysis, where the analyses are conducted either under LRFD load combinations directly or under ASD load combinations with a 1.6 factor applied to the ASD gravity loads.

3. A reduced flexural stiffness, EI^* ,

$$EI^* = 0.8\tau_b EI \quad (\text{A-7-2})$$

shall be used for all members whose flexural stiffness is considered to contribute to the lateral stability of the structure,

where

$$\begin{aligned} I &= \text{moment of inertia about the axis of bending, mm}^4 \\ \tau_b &= 1.0 \text{ for } \alpha P_r/P_y \leq 0.5 \\ &= 4[\alpha P_r/P_y(1 - \alpha P_r/P_y)] \text{ for } \alpha P_r/P_y > 0.5 \\ P_r &= \text{required axial compressive strength under LRFD or ASD load combinations, N} \\ P_y &= AF_y, \text{ member yield strength, N} \end{aligned}$$

$$\text{and } \alpha = 1.0(\text{LRFD}) \quad \alpha = 1.6(\text{ASD})$$

In lieu of using $\tau_b < 1.0$ where $\alpha P_r/P_y \leq 0.5$, $\tau_b = 1.0$ may be used for all members, provided that an additive notional load of $0.001Y_t$ is added to the notional load required in (2).

4. A reduced axial stiffness, EA^* ,

$$EA^* = 0.8EA \quad (\text{A-7-3})$$

shall be used for members whose axial stiffness is considered to contribute to the lateral stability of the structure, where A is the cross-sectional member area.

PART 2A

SEISMIC PROVISION FOR STRUCTURAL STEEL BUILDINGS

SYMBOLS

A_b	= Cross-sectional area of a horizontal boundary element (HBE), mm ²
A_c	= Cross-sectional area of a vertical boundary element (VBE), mm ²
A_f	= Flange area, mm ²
A_g	= Gross area, mm ²
A_{sc}	= Area of the yielding segment of steel core, mm ²
A_{sh}	= Minimum area of tie reinforcement, mm ²
A_{sp}	= Horizontal area of the steel plate in composite shear wall, mm ²
A_{st}	= Area of link stiffener, mm ²
A_w	= Link web area, mm ²
C_a	= Ratio of required strength to available strength
C_d	= Coefficient relating relative brace stiffness and curvature
C_d	= Deflection amplification
C_r	= Parameter used for determining the approximate fundamental period
D	= Dead load due to the weight of the structural elements and permanent features on the building
D	= Outside diameter of round HSS, mm
E	= Earthquake load
E	= Effect of horizontal and vertical earthquake-induced loads
E	= Modulus of elasticity of steel, 200,000 MPa
EI	= Flexural elastic stiffness of the chord members of the special segment, N-mm ²
F_y	= Specified minimum yield stress of the type of steel to be used, MPa. As used in the Specification, "yield stress" denotes either the minimum specified yield point (for those steels that have a yield point) or the specified yield strength (for those steels that do not have a yield point)
F_{yb}	= F_y of a beam, MPa
F_{yc}	= F_y of a column, MPa
F_{yh}	= Specified minimum yield stress of the ties, MPa
F_{ysc}	= Specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, MPa
F_u	= Specified minimum tensile strength, MPa

H	= Height of story, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, mm	P_a	= Required axial strength of a column using ASD load combinations, N
I	= Moment of inertia, mm ⁴	P_{ac}	= Required compressive strength using ASD load combinations, N
I_c	= Moment of inertia of a vertical boundary element (VBE) taken perpendicular to the direction of the web plate line, mm ⁴	P_b	= Required strength of lateral brace at ends of the link, N
K	= Effective length factor for prismatic member	P_c	= Available axial strength of a column, N
L	= Live load due to occupancy and moveable equipment, kN	P_n	= Nominal axial strength of a column, N
L	= Span length of the truss, mm	P_n	= Nominal compressive strength of the composite column calculated in accordance with the Specification, N
L	= Distance between VBE centerlines, mm	P_{nc}	= Nominal axial compressive strength of diagonal members of the special segment, N
L_b	= Length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm	P_{nt}	= Nominal axial tensile strength of diagonal members of the special segment, N
L_b	= Link length, mm	P_o	= Nominal axial strength of a composite column at zero eccentricity, N
L_{cf}	= Clear distance between VBE flanges, mm	P_r	= Required compressive strength, N
L_h	= Distance between plastic hinge locations, mm	P_u	= Required axial strength of a column or a link using LRFD load combinations, N
L_p	= Limiting laterally unbraced length for full plastic flexural strength, uniform moment case, mm	P_u	= Required axial strength of a composite column, N
L_{pd}	= Limiting laterally unbraced length for plastic analysis, mm	P_{uc}	= Required compressive strength using LRFD load combinations, N
L_s	= Length of the special segment, mm	P_y	= Nominal axial yield strength of a member, equal to $F_y A_g$, N
M_a	= Required flexural strength, using ASD load combinations, N-mm	P_{ysc}	= Axial yield strength of steel core, N
M_{av}	= Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on ASD load combinations, N-mm	Q_b	= Maximum unbalanced vertical load effect applied to a beam by the braces, N
M_n	= Nominal flexural strength, N-mm	Q_1	= Axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link
M_{nc}	= Nominal flexural strength of the chord member of the special segment, N-mm	R	= Seismic response modification coefficient
M_p	= Nominal plastic flexural strength, N-mm	R_n	= Nominal strength, N
M_{pa}	= Nominal plastic flexural strength modified by axial load, N-mm	R_t	= Ratio of the expected tensile strength to the specified minimum tensile strength F_u , as related to over strength in material yield stress
M_{pb}	= Nominal plastic flexural strength of the beam, N-mm	R_y	= Required strength
$M_{p,exp}$	= Expected plastic moment, N-mm	R_u	= Panel zone nominal shear strength
M_{pc}	= Nominal plastic flexural strength of the column, N-mm	R_v	= Ratio of the expected yield stress to the specified minimum yield stress, F_y
M_r	= Expected flexural strength, N-mm	R_y	= Required shear strength using ASD load combinations, N
M_{uv}	= Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRFD load combinations, N-mm	V_a	= Nominal shear strength of a member, N
M_u	= Required flexural strength, using LRFD load combinations, N-mm	V_n	= Expected vertical shear strength of the special segment, N
$M_{u,exp}$	= Expected required flexural strength, N-mm	V_{ne}	= Nominal shear strength of the steel plate in a composite plate shear wall, N
		V_{ns}	= Nominal shear strength of an active link, N
		V_p	= Nominal shear strength of an active link modified by the axial load magnitude, N
		V_{pa}	

V_u	= Required shear strength using LRFD load combinations, N	t	= Thickness of element, mm
Y_{con}	= Distance from top of steel beam to top of concrete slab or encasement, mm	t	= Thickness of column web or doubler plate, mm
Y_{PNA}	= Maximum distance from the maximum concrete compression fiber to the plastic neutral axis, mm	t_{bf}	= Thickness of beam flange, mm
Z	= Plastic section modulus of a member, mm ³	t_{cf}	= Thickness of column flange, mm
Z_b	= Plastic section modulus of the beam, mm ³	t_f	= Thickness of flange, mm
Z_c	= Plastic section modulus of the column, mm ³	t_{min}	= Minimum wall thickness of concrete-filled rectangular HSS, mm
Z_x	= Plastic section modulus x-axis, mm ³	t_p	= Thickness of panel zone including doubler plates, mm
Z_{RBS}	= Minimum plastic section modulus at the reduced beam section, mm ³	t_w	= Thickness of web, mm
b	= Width of compression element as defined in Specification Section 502.4.1, mm	w_z	= Width of panel zone between column flanges, mm
b_{cf}	= Width of column flange, mm	x	= Parameter used for determining the approximate fundamental period (I-R2)
b_f	= Flange width, mm	z_b	= Minimum plastic section modulus at the reduced beam section, mm ³
b_w	= Width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear, mm	ΣM_{pc}	= Moment at beam and column centerline determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g , from the top and bottom of the beam moment connection
d	= Nominal fastener diameter, mm	ΣM_{pb}	= Moment at the intersection of the beam and column centerlines determined by projecting the beam maximum developed moments from the column face. Maximum developed moments shall be determined from test results
d	= Overall beam depth, mm	β	= Compression strength adjustment factor
d_c	= Overall column depth, mm	Δ	= Design story drift
d_z	= Overall panel zone depth between continuity plates, mm	Δ_b	= Deformation quantity used to control loading of test specimen (total brace end rotation for the subassembly test specimen; total brace axial Deformation for the brace test specimen)
e	= EBF link length, mm	Δ_{bm}	= Value of deformation quantity,
f'_c	= Specified compressive strength of concrete, MPa	Δ_b	= corresponding to the design story drift
h	= Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; and for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, mm	Δ_{by}	= Value of deformation quantity, Δ_b , at first significant yield of test specimen
h	= Distance between horizontal boundary element centerlines, mm	Ω	= Safety factor
h_{cc}	= Cross-sectional dimension of the confined core region in composite columns measured center-to-center of the transverse reinforcement, mm	Ω_b	= Safety factor for flexure = 1.67
h_o	= Distance between flange centroids, mm	Ω_c	= Safety factor for compression = 1.67
l	= Unbraced length between stitches of built-up bracing members, mm	Ω_o	= Horizontal seismic overstrength factor
l	= Unbraced length of compression or bracing member, mm	Ω_v	= Safety factor for shear strength of panel zone of beam-to-column connections
r	= Governing radius of gyration, mm	α	= Angle of diagonal members with the horizontal
r_y	= Radius of gyration about y-axis, mm	α	= Angle of web yielding in radians, as measured relative to the vertical
s	= Spacing of transverse reinforcement measured along the longitudinal axis of the structural composite member, mm	δ	= Deformation quantity used to control loading of test specimen
t	= Thickness of connected part, mm	δ_y	= Value of deformation quantity δ at first significant yield of test specimen

ρ'	=	Ratio of required axial force P_u to required shear strength V_u of a link
λ_p, λ_{ps}	=	Limiting slenderness parameter for compact element
ϕ	=	Resistance factor
ϕ_b	=	Resistance factor for flexure
ϕ_c	=	Resistance factor for compression
ϕ_v	=	Resistance factor for shear strength of panel zone of beam-to-column connections
ϕ_v	=	Resistance factor for shear
ϕ_v	=	Resistance factor for the shear strength of a composite column
θ	=	Interstory drift angle, radians
γ_{total}	=	Link rotation angle
ω	=	Strain hardening adjustment factor

PART 2B

SECTION 514

STRUCTURAL STEEL BUILDING PROVISIONS

514.1 Scope

The Seismic Provisions for Structural Steel Buildings, hereinafter referred to as these Provisions, shall govern the design, fabrication and erection of structural steel members and connections in the seismic load resisting systems (SLRS) and splices in columns that are not part of the SLRS, in buildings and other structures, where other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting-elements. These Provisions shall apply when the seismic response modification coefficient, R , (as specified in the NSCP code) is taken greater than 3, regardless of the seismic design category. When the seismic response modification coefficient, R , is taken as 3 or less, the structure is not required to satisfy these Provisions, unless specifically required by the NSCP code.

These Provisions shall be applied in conjunction with Chapter 5 Steel and Metal, hereinafter referred to as the Specification. Members and connections of the SLRS shall satisfy the requirements of the NSCP code, the Specification, and these Provisions.

Wherever these provisions refer to the NSCP code and there is no local building code, the loads, load combinations, system limitations and general design requirements shall be those in SEI/ASCE 7.

User Note: The NSCP code generally restricts buildings designed with an R factor of 3 or less to seismic design categories (SDC) A, B or C; however, some systems such as cantilever columns that have R factors less than 3 are permitted in SDC D and above and these Provisions apply. See the NSCP code for specific system limitations.

Part 2A includes a Glossary that is specifically applicable to this Part, and Section B-1, B-2, B-3, B-4, B-5, B-6 and B-7

SECTION 515 REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

The documents referenced in these Provisions shall include those listed in Specification Section 501.2 with the following additions and modifications:

American Institute of Steel Construction (AISC)

Specification for Structural Steel Buildings, ANSI/AISC 360-05 Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, ANSI/AISC 358-05

American Society for Nondestructive Testing (ASNT)

Recommended Practice for the Training and Testing of Nondestructive Testing Personnel, ASNT SNT TC-1a-2001

Standard for the Qualification and Certification of Nondestructive Testing Personnel, ANSI/ASNT CP-189-2001

American Welding Society (AWS)

Standard Methods for Determination of the Diffusible Hydrogen Content of Martensitic, Bainitic, and Ferritic Steel Weld Metal Produced by Arc Welding, AWS A4.3-93R

Standard Methods for Mechanical Testing of Welds-U.S. Customary, ANSI/ AWS B4.0-98

Standard Methods for Mechanical Testing of Welds-Metric Only, ANSI/AWS B4.0M:2000

Standard for the Qualification of Welding Inspectors, AWS B5.1:2003

Describing Oxygen-Cut Surfaces, AWS C4.1

Federal Emergency Management Agency (FEMA)

Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, FEMA 350, July 2000

DAO 07:2008 Mandatory PNS for Equal Leg Angle Beams

DAO 05:2002 Mandatory PNS for Steel Bars

SECTION 516 GENERAL SEISMIC DESIGN REQUIREMENTS

The required strength and other seismic provisions for seismic Zones 2 and 4 including limitations on height and irregularity shall be as specified in the NSCP code.

The design story drift shall be determined as required in the NSCP code.

SECTION 517 LOADS, LOAD COMBINATIONS, AND NOMINAL STRENGTHS

517.1 Loads and Load Combinations

The loads and load combinations shall be as stipulated by the NSCP code. Where amplified seismic loads are required by these Provisions, the horizontal portion of the earthquake load E (as defined in the NSCP code) shall be multiplied by the overstrength factor, Ω_o , prescribed by the NSCP code.

517.2 Nominal Strength

The nominal strength of systems, members and connections shall comply with the Specification, except as modified throughout these Section.

SECTION 518 STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS, SHOP DRAWINGS, AND ERECTION DRAWINGS

518.1 Structural Design Drawings and Specifications

Structural design drawings and specifications shall show the work to be performed, and include items required by the Specification and the following, as applicable:

1. Designation of the seismic load resisting system (SLRS)
2. Designation of the members and connections that are part of the SLRS
3. Configuration of the connections
4. Connection material specifications and sizes
5. Locations of demand critical welds
6. Lowest anticipated service temperature (LAST) of the steel structure, if the structure is not enclosed and maintained at a temperature of 10 °C or higher
7. Locations and dimensions of protected zones
8. Locations where gusset plates are to be detailed to accommodate inelastic rotation
9. Welding requirements as specified in Section B-6, Section B-6.2.

User Note: These Provisions should be consistent with the Code of Standard Practice, as designated in Section 501.4 of the Specification. There may be specific connections and applications for which details are not specifically addressed by the Provisions. If such a condition exists, the contract documents should include appropriate requirements for those applications. These may include nondestructive testing requirements beyond those in Section B-2, bolt hole fabrication requirements beyond those permitted by the Specification, bolting requirements other than those in the Research Council on Structural Connections (RCSC) Specification for Structural Joints Using ASTM A325 or A490 Bolts, or welding requirements other than those in Section B-6.

518.2 Shop Drawings

Shop drawings shall include items required by the Specification and the following, as applicable:

1. Designation of the members and connections that are part of the SLRS
2. Connection material specifications
3. Locations of demand critical shop welds
4. Locations and dimensions of protected zones
5. Gusset plates drawn to scale when they are detailed to accommodate inelastic rotation
6. Welding requirements as specified in Section B-6, Section B-2.2.

User Note: There may be specific connections and applications for which details are not specifically addressed by the section. If such a condition exists, the shop drawings should include appropriate requirements for that application. These may include bolt hole fabrication requirements beyond those permitted by the Specification, bolting requirements other than those in the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, and welding requirements other than those in Section B-6. See Section 513 of the Specification for additional provisions on shop drawings.

518.3 Erection Drawings

Erection drawings shall include items required by the Specification and the following, as applicable:

1. Designation of the members and connections that are part of the SLRS
2. Field connection material specifications and sizes
3. Locations of demand critical field welds
4. Locations and dimensions of protected zones
5. Locations of pretensioned bolts
6. Field welding requirements as specified in Section B-6, Section B-2.3

User Note: There may be specific connections and applications for which details are not specifically addressed by the Provisions. If such a condition exists, the erection drawings should include appropriate requirements for that application. These may include bolting requirements other than those in the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, and welding requirements other than those in Section B-6. See Section M1 of the Specification for additional provisions on erection drawings.

SECTION 519 MATERIALS

519.1 Material Specifications

Structural steel used in the seismic load resisting system (SLRS) shall meet the requirements of Specification Section 501.3.1.1, except as modified in these Provisions. The specified minimum yield stress of steel to be used for members in which inelastic behavior is expected shall not exceed 345 MPa for systems defined in Sections 522, 523, 525, 526, 528, 529, and 530 nor 380 MPa for systems defined in Sections 524 and 527, unless the suitability of the material is determined by testing or other rational criteria. This limitation does not apply to columns for which the only expected inelastic behavior is yielding at the column base.

The structural steel used in the SLRS described in Sections 522, 523, 524, 525, 526, 527, 528, 529 and 530 shall meet one of the following ASTM Specifications: A36/ A36M, A53/A53M, A500 (Grade B or C), A501, A529/A529M, A572/A572M [Grade 42 (290 Mpa), 50 (345 Mpa) or 55 (380 Mpa)], A588/A588M, A913/A913M [Grade 50 (345 Mpa), 60 (415 Mpa) or 65 (450 Mpa)], A992/A992M, or A1011 HSLAS Grade 55 (380 Mpa). The structural steel used for column base plates shall meet one of the preceding ASTM specifications or ASTM A283/A283M Grade D.

Other steels and non-steel materials in buckling-restrained braced frames are permitted to be used subject to the requirements of Section 529 and Section B-5.

User Note: This section only covers material properties for structural steel used in the SLRS and included in the definition of structural steel given in Section 2.1 of the AISC Code of Standard Practice. Other steel, such as cables for permanent bracing is not included.

519.2 Material Properties for Determination of Required Strength of Members and Connections

When required in these Provisions, the required strength of an element (a member or a connection) shall be determined from the expected yield stress, $R_y F_y$, of an adjoining member, where F_y is the specified minimum yield stress of the grade of steel to be used in the adjoining members and R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y , of that material.

Table 519-1 R_y and R_t Values for Different Member Types

Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/36M	1.5	1.2
• ASTM A572/572M Grade 290	1.3	1.1
• ASTM A572/572M Grade 345 or 380	1.1	1.1
• ASTM A913/A913M Grade 345, 415, 450		
• ASTM A588/A588M		
• ASTM A992/A992M, A1011		
• HSLAS Grade 380		
• ASTM A529 Grade 345	1.2	1.2
• ASTM A529 Grade 380	1.1	1.2
Hollow structural sections (HSS)		
• ASTM A500 (Grade B or C), ASTM A501	1.4	1.3
Pipe		
• ASTM A53/A53M	1.6	1.2
Plates		
• ASTM A36/A36M	1.3	1.2
• ASTM A572/A572M Grade 345	1.1	1.2
• ASTM A588/A588M		

The available strength of the element, ϕR_n for LRFD and R_n/Ω for ASD, shall be equal to or greater than the required strength, where R_n is the nominal strength of the connection. The expected tensile strength, $R_t F_u$, and the expected yield stress, $R_y F_y$, are permitted to be used in lieu of F_u and F_y , respectively, in determining the nominal strength, R_n , of rupture and yielding limit states within the same member for which the required strength is determined.

User Note: In several instances a member, or a connection limit state within that member, is required to be designed for forces corresponding to the expected strength of the member itself. Such cases include brace fracture limit states (block shear rupture and net section fracture in the brace in SCBF), the design of the beam outside of the link in EBF, etc. In such cases it is permitted to use the expected material strength in the determination of available member strength. For connecting elements and for other members, specified material strength should be used.

The values of R_y and R_t for various steels are given in Table 519-1. Other values of R_y and R_t shall be permitted if the values are determined by testing of specimens similar in size and source conducted in accordance with the requirements for the specified grade of steel.

519.3 Heavy Section CVN Requirements

For structural steel in the SLRS, in addition to the requirements of Specification Section 501.3.1c, hot rolled shapes with flanges 38 mm thick and thicker shall have a minimum Charpy V-Notch toughness of 20 ft-lb (27 J) at 70 °F (21 °C), tested in the alternate core location as described in ASTM A6 Supplementary Requirement S30. Plates 50 mm thick and thicker shall have a minimum Charpy V-Notch toughness of 20 ft-lb (27 J) at 70 °F (21 °C), measured at any location permitted by ASTM A673, where the plate is used in the following:

1. Members built-up from plate
2. Connection plates where inelastic strain under seismic loading is expected
3. At the steel core of buckling-restrained braces

User Note: Examples of connection plates where inelastic behavior is expected include, but are not limited to, gusset plates intended to function as a hinge and allow out-of-plane buckling of braces, some bolted flange plates for moment connections, some end plates for bolted moment connections, and some column base plates designed as a pin

SECTION 520

CONNECTIONS, JOINTS, AND FASTENERS

520.1 Scope

Connections, joints and fasteners that are part of the seismic load resisting system (SLRS) shall comply with Specification Section 510, and with the additional requirements of this Section.

The design of connections for a member that is a part of the SLRS shall be configured such that a ductile limit state in either the connection or the member controls the design.

User Note: An example of a ductile limit state is tension yielding. It is unacceptable to design connections for members that are part of the SLRS such that the strength limit state is governed by non-ductile or brittle limit states, such as fracture, in either the connection or the member.

520.2 Bolted Joints

All bolts shall be pretensioned high strength bolts and shall meet the requirements for slip-critical faying surfaces in accordance with Specification Section 510.1.8 with a Class A surface. Bolts shall be installed in standard holes or in short-slotted holes perpendicular to the applied load. For brace diagonals, oversized holes shall be permitted when the connection is designed as a slip-critical joint, and the oversized hole is in one ply only. Alternative hole types are permitted if designated in the Prequalified Connections for Special and Intermediate Moment Frames for Seismic Applications (ANSI/AISC 358), or if otherwise determined in a connection prequalification in accordance with Section B-1, or if determined in a program of qualification testing in accordance with Section B-4 or B-5. The available shear strength of bolted joints using standard holes shall be calculated as that for bearing-type joints in accordance with Specification Sections 510.1.3 and 510.1.10, except that the nominal bearing strength at bolt holes shall not be taken greater than $2.4d_tF_u$.

Exception:

The faying surfaces for end plate moment connections are permitted to be coated with coatings not tested for slip resistance or with coatings with a slip coefficient less than that of a Class A faying surface.

Bolts and welds shall not be designed to share force in a joint or the same force component in a connection.

User Note: A member force, such as a brace axial force, must be resisted at the connection entirely by one type of joint (in other words, either entirely by bolts or entirely by welds). A connection in which bolts resist a force that is normal to the force resisted by welds, such as a moment connection in which welded flanges transmit flexure and a bolted web transmits shear, is not considered to be sharing the force.

520.3 Welded Joints

Welding shall be performed in accordance with Section B-6. Welding shall be performed in accordance with a welding procedure specification (WPS) as required in AWS D1.1 and approved by the engineer-of-record. The WPS variables shall be within the parameters established by the filler metal manufacturer.

520.3.1 General Requirements

All welds used in members and connections in the SLRS shall be made with a filler metal that can produce welds that have a minimum Charpy V-Notch toughness of 20 ft-lb (27 J) at 0 °F (minus 18 °C), as determined by the appropriate AWS A5 classification test method or manufacturer certification. This requirement for notch toughness shall also apply in other cases as required in these Provisions.

520.3.2 Demand Critical Welds

Where welds are designated as demand critical, they shall be made with a filler metal capable of providing a minimum Charpy V-Notch (CVN) toughness of 27 J at 29 °C as determined by the appropriate AWS classification test method or manufacturer certification, and 54 J at 21 °C as determined by Section B-7 or other approved method, when the steel frame is normally enclosed and maintained at a temperature of 10 °C or higher. For structures with service temperatures lower than 10 °C, the qualification temperature for Section B-7 shall be 11 °C above the lowest anticipated service temperature, or at a lower temperature.

SMAW electrodes classified in AWS A5.1 as E7018 or E7018-X, SMAW electrodes classified in AWS A5.5 as E7018-C3L or E8018-C3, and GMAW solid electrodes are exempted from production lot testing when the CVN toughness of the electrode equals or exceeds 27 J at a temperature not exceeding 29 °C as determined by AWS classification test methods. The manufacturer's certificate of compliance shall be considered sufficient evidence of meeting this requirement.

User Note: Welds designated demand critical are specifically identified in the Provisions in the section applicable to the designated SLRS.

There may be specific welds similar to those designated as demand critical by these Provisions that have not been specifically identified as demand critical by these Provisions that warrant such designation. Consideration of the demand critical designation for such welds should be based upon the inelastic strain demand and the consequence of failure. Complete-joint-penetration (CJP) groove welds between columns and base plates should be considered demand critical similar to column splice welds, when CJP groove welds used for column splices in the designated SLRS have been designated demand critical.

For special and intermediate moment frames, typical examples of demand critical welds include the following CJP groove welds:

1. Welds of beam flanges to columns
2. Welds of single plate shear connections to columns
3. Welds of beam webs to columns
4. Column splice welds, including column bases

For ordinary moment frames, typical examples include CJP groove welds in items 1, 2, and 3 above.

For Eccentrically Braced Frames (EBF), typical examples of demand critical welds include CJP groove welds between link beams and columns. Other welds, such as those joining the web plate to flange plates in built-up EBF link beams, and column splice welds when made using CJP groove welds, should be considered for designation as demand critical welds.

520.3.3 Recommended Joint

The use of Type I welded joints is not allowed in seismic Zone 4. Type II joints are recommended as in the use of Proprietary Welded Joint.

520.4 Protected Zone

Where a protected zone is designated by these Provisions or ANSI/AISC 358, it shall comply with the following:

1. Within the protected zone, discontinuities created by fabrication or erection operations, such as tack welds, erection aids; air-arc gouging and thermal cutting shall be repaired as required by the engineer-of-record.
2. Welded shear studs and decking attachments that penetrate the beam flange shall not be placed on beam flanges within the protected zone. Decking arc spot welds as required to secure decking shall be permitted.

3. Welded, bolted, screwed or shot-in attachments for perimeter edge angles, exterior facades, partitions, duct work, piping or other construction shall not be placed within the protected zone.

Exception:

Welded shear studs and other connections shall be permitted when designated in the Prequalified Connections for Special and Intermediate Moment Frames for Seismic Applications (ANSI/AISC 358), or as otherwise determined in accordance with a connection prequalification in accordance with Section B-1, or as determined in a program of qualification testing in accordance with Section B-4.

Outside the protected zone, calculations based upon the expected moment shall be made to demonstrate the adequacy of the member net section when connectors that penetrate the member are used.

520.5 Continuity Plates and Stiffeners

Corners of continuity plates and stiffeners placed in the webs of rolled shapes shall be clipped as described below. Along the web, the clip shall be detailed so that the clip extends a distance of at least 38 mm beyond the published k detail dimension for the rolled shape. Along the flange, the clip shall be detailed so that the clip does not exceed a distance of 12 mm beyond the published k_1 detail dimension. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. If a curved clip is used, it shall have a minimum radius of 12 mm.

At the end of the weld adjacent to the column web/flange juncture, weld tabs for continuity plates shall not be used, except when permitted by the engineer-of-record. Unless specified by the engineer-of-record that they be removed, weld tabs shall not be removed when used in this location.

SECTION 521

MEMBERS

521.1 Scope

Members in the seismic load resisting system (SLRS) shall comply with the Specification and Section 521. For columns that are not part of the SLRS, see Section 521.1.2.

521.2 Classification of Sections for Local Buckling

521.2.1 Compact

When required by these Provisions, members of the SLRS shall have flanges continuously connected to the web or webs and the width-thickness ratios of its compression elements shall not exceed the limiting width-thickness ratios, λ_p , from Specification Table 502.4.1.

521.2.2 Seismically Compact

When required by these Provisions, members of the SLRS must have flanges continuously connected to the web or webs and the width-thickness ratios of its compression elements shall not exceed the limiting width-thickness ratios, λ_{ps} , from Provisions Table 521-1.

Table 521-1 Limiting Width-Thickness Ratios for Compression Elements

Description of Element		Width – Thickness Ratio	Limiting Width-Thickness Ratios
			λ_{ps} Seismically Compact
Unstiffened Elements	Flexure in flanges of rolled or built-up I-shaped sections [a],[c],[e],[g],[h]	b/t	$0.30 \sqrt{E/F_y}$
	Uniform compression in flanges of rolled or built-up I-shaped sections [b],[h]	b/t	$0.30 \sqrt{E/F_y}$
	Uniform compression in flanges of rolled or built-up I-shaped sections [d]	b/t	$0.38 \sqrt{E/F_y}$
	Uniform compression in flanges of channels, outstanding legs of pairs of angles in continuous contact, and braces [c],[g]	b/t	$0.30 \sqrt{E/F_y}$
	Uniform compression in flanges of H-pile sections	b/t	$0.45 \sqrt{E/F_y}$
	Flat Bars [f]	b/t	2.5
	Uniform compression in legs of single angles, legs of double angle members with separators, or flanges of tees [g]	b/t	$0.30 \sqrt{E/F_y}$
	Uniform compression in stems of tees [g]	b/t	$0.30 \sqrt{E/F_y}$
Stiffened Elements	Webs in flexural compression in beams in SMF, Section 522, unless noted otherwise	h/t_w	$2.45 \sqrt{E/F_y}$
	Webs in flexural compression or combined flexure and axial compression [a],[c],[g],[h],[i],[j]	h/t_w	For $C_a \leq 0.125$ [k] $3.14 \sqrt{E/F_y} (1 - 1.54C_a)$

			For $C_a \leq 0.125$ [k]
		h/t_w	$1.14 \sqrt{E/F_y} (2.33 C_a) \geq 1.49 \sqrt{E/F_y}$
	Round HSS in axial and/or flexural compression [c],[g]	D/t	$0.044 \sqrt{E/F_y}$
	Rectangular HSS in axial and/or flexural compression [c],[g]	b/t or h/t_w	$0.64 \sqrt{E/F_y}$
	Webs of H-Pile sections	h/t_w	$0.94 \sqrt{E/F_y}$
[a] Required for beams in SMF Section 522 and SPSW Section 530 [b] Required for columns in SMF Section 522, unless the ratios from Eq. 522-3 are greater than 2.0 where it is permitted to use λ_p in specification Table 502.4.1 [c] Required for braces and columns in SCBF Section 526 and braces in OCBF Section 527 [d] It is permitted to use λ_p in Specification Table 502.4.1 for columns in STMF Section 522 and columns in EBF Section 528 [e] Required for link in EBF Section 528 except it is permitted to use λ_p in Table 502.4.1 of the specification for flanges of links of length $1.6 M_p/V_p$ are defined in Section 528 [f] Diagonal web members within the special segment of STMF Section 525 [g] Chord members of STMF Section 525 [h] Required for beams and columns in BRBF Section 529 [i] Required for columns in SPSW Section 530 [j] For columns in STMF Section 522 columns in SMF, if the ratios from Eq. 522-3 are greater than 2.0; for columns in EBF Section 528; or EBF webs of links of length $1.6 M_p/V_p$ or less, it is permitted to use the following for λ_p For $C_a \leq 0.125$, $\lambda_p = 3.76 \sqrt{E/F_y} (1 - 2.75 C_a)$ For $C_a > 0.125$, $\lambda_p = 3.76 \sqrt{E/F_y} (2.33 - C_a) \geq 1.49 \sqrt{E/F_y}$ [k] for LRFD, $C_a = (P_u / \phi_b P_y)$ for ASD, $C_a = (\Omega_b P_a / P_y)$ where P_a = required compressive strength (ASD), N P_u = required compressive strength (LRFD), N P_y = axial yield strength, N $\phi_b = 0.90$ $\Omega_b = 1.67$			

521.3 Column Strength

When $P_u/\phi P_n (LRFD) > 0.4$ or $\Omega_c P_a/P_n (ASD) > 0.4$, as appropriate, without consideration of the amplified seismic load,

where

$\phi_c = 0.90 (LRFD)$ $\Omega_c = 1.67 (ASD)$
 P_a = required axial strength of a column using ASD Load combinations, N
 P_n = nominal axial strength of a column, N
 P_u = required axial strength of a column using LRFD load combinations, N

The following requirement shall be met:

1. The required axial compressive and tensile strength, considered in the absence of any applied moment, shall be determined using the load combinations stipulated by the NSCP code including the amplified seismic load
2. The required axial compressive and tensile strength shall not exceed either of the following:
 - a. The maximum load transferred to the column considering $1.1R_y (LRFD)$ or $(1.1/1.5)R_y (ASD)$, as appropriate, times the nominal strengths of the connecting beam or brace elements of the building.
 - b. The limit as determined from the resistance of the foundation to overturning uplift.

521.4 Column Splices

521.4.1 General

The required strength of column splices in the seismic load resisting system (SLRS) shall equal the required strength of the columns, including that determined from Sections 521.3, 522.9, 523.9, 524.9, 526.5 and 529.5.2.

In addition, welded column splices that are subject to a calculated net tensile load effect determined using the load combinations stipulated by the NSCP code including the amplified seismic load, shall satisfy both of the following requirements:

1. The available strength of partial-joint-penetration (PJP) groove welded joints, if used, shall be at least equal to 200 percent of the required strength.

2. The available strength for each flange splice shall be at least equal to $0.5R_y F_y A_f (LRFD)$ or $(0.5/1.5)R_y F_y A_f (ASD)$, as appropriate, where $R_y F_y$ is the expected yield stress of the column material and A_f is the flange area of the smaller column connected.

Beveled transitions are not required when changes in thickness and width of flanges and webs occur in column splices where PJP groove welded joints are used.

Column web splices shall be either bolted or welded, or welded to one column and bolted to the other. In moment frames using bolted splices, plates or channels shall be used on both sides of the column web.

The centerline of column splices made with fillet welds or partial-joint-penetration groove welds shall be located 1.2 m or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 2.4 m, splices shall be at half the clear height.

521.4.2 Columns Not Part of the Seismic Load Resisting System

Splices of columns that are not a part of the SLRS shall satisfy the following:

1. Splices shall be located 1.2 m or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 2.4 m, splices shall be at half the clear height.
2. The required shear strength of column splices with respect to both orthogonal axes of the column shall be $M_{pc}/H (LRFD)$ or $M_{pc}/1.5H (ASD)$, as appropriate, where M_{pc} is the lesser nominal plastic flexural strength of the column sections for the direction in question, and H is the storey height.

521.5 Column Bases

The required strength of column bases shall be calculated in accordance with Sections 521.5.1, 521.5.2, and 521.5.3. The available strength of anchor rods shall be determined in accordance with Specification Section 510.3.

The available strength of concrete elements at the column base, including anchor rod embedment and reinforcing steel, shall be in accordance with ACI 318, Appendix D.

User Note: When using concrete reinforcing steel as part of the anchorage embedment design, it is important to understand the anchor failure modes and provide reinforcement that is developed on both sides of the expected failure surface. See ACI 318, Appendix D, Figure RD.4.1 and Section D.4.2.1, including Commentary for additional information.

Exception:

The special requirements in ACI 318, Appendix D, for "regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories" need not be applied.

521.5.1 Required Axial Strength

The required axial strength of column bases, including their attachment to the foundation, shall be the summation of the vertical components of the required strengths of the steel elements that are connected to the column base.

521.5.2 Required Shear Strength

The required shear strength of column bases, including their attachments to the foundations, shall be the summation of the horizontal component of the required strengths of the steel elements that are connected to the column base as follows:

1. For diagonal bracing, the horizontal component shall be determined from the required strength of bracing connections for the seismic load resisting system (SLRS).
2. For columns, the horizontal component shall be at least equal to the lesser of the following:

- a. $2R_y F_y Z_x / H$ (LRFD) or $(2/1.5) R_y F_y Z_x / H$ (ASD), as appropriate of the column.

Where

H = height of storey, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, mm

- b. The shear calculated using the load combinations of the NSCP code, including the amplified seismic load.

521.5.2 Required Flexural Strength

The required flexural strength of column bases, including their attachment to the foundation, shall be the summation of the required strengths of the steel elements that are connected to the column base as follows:

1. For diagonal bracing, the required flexural strength shall be at least equal to the required strength of bracing connections for the SLRS.
2. For columns, the required flexural strength shall be at least equal to the lesser of the following:
 - a. $1.1 R_y F_y Z$ (LRFD) or $(1.1/1.5) R_y F_y Z$ (ASD) as appropriate, of the column or
 - b. the moment calculated using the load combinations of the NSCP code, including the amplified seismic load.

521.6 H-Piles

521.6.1 Design of H-Piles

Design of H-piles shall comply with the provisions of the Specification regarding design of members subjected to combined loads. H-piles shall meet the requirements of Section 521.2.2.

521.6.2 Battered H-Piles

If battered (sloped) and vertical piles are used in a pile group, the vertical piles shall be designed to support the combined effects of the dead and live loads without the participation of the battered piles.

521.6.3 Tension in H-Pile

Tension in each pile shall be transferred to the pile cap by mechanical means such as shear keys, reinforcing bars or studs welded to the embedded portion of the pile. Directly below the bottom of the pile cap, each pile shall be free of attachments and welds for a length at least equal to the depth of the pile cross section.

SECTION 522

SPECIAL MOMENT FRAMES (SMF)

522.1 Scope

Special moment frames (SMF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. SMF shall satisfy the requirements in this Section.

522.2 Beam-to-Column Connections

522.2.1 Requirements

Beam-to-column connections used in the seismic load resisting system (SLRS) shall satisfy the following three requirements:

1. The connection shall be capable of sustaining an interstory drift angle of at least 0.04 radians.
2. The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80 M_p$ of the connected beam at an interstory drift angle of 0.04 radians.
3. The required shear strength of the connection shall be determined using the following quantity for the earthquake load effect E :

$$E = 2[1.1R_y M_p]/L_h \quad (522-1)$$

where

R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y

M_p = nominal plastic flexural strength, N-mm

L_h = distance between plastic hinge locations, mm

When E as defined in Eq. 522-1 is used in ASD load combinations that are additive with other transient loads and that are based on SEI/ASCE 7, the 0.75 combination factor for transient loads shall not be applied to E .

Connections that accommodate the required interstory drift angle within the connection elements and provide the measured flexural resistance and shear strengths specified above are permitted. In addition to satisfying the requirements noted above, the design shall demonstrate that any additional drift due to connection deformation can be accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.

522.2.2 Conformance Demonstration

Beam-to-column connections used in the SLRS shall satisfy the requirements of Section 522.2 by one of the following:

1. Use of SMF connections designed in accordance with ANSI/AISC 358.
2. Use of a connection prequalified for SMF in accordance with Section B-1.
3. Provision of qualifying cyclic test results in accordance with Section B-4. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:
 - a. Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section B-4.
 - b. Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Section B-4.

522.2.3 Welds

Unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Section B-1, or as determined in a program of qualification testing in accordance with Section B-4, complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be demand critical welds as described in Section 520.3.2.

User Note: For the designation of demand critical welds, standards such as ANSI/AISC 358 and tests addressing specific connections and joints should be used in lieu of the more general terms of these Provisions. Where these Provisions indicate that a particular weld is designated demand critical, but the more specific standard or test does not make such a designation, the more specific standard or test should govern. Likewise, these standards and tests may designate welds as demand critical that are not identified as such by these Provisions.

522.2.4 Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall meet the requirements of Section 520.4. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section B-1, or as determined in a program of qualification testing in accordance with Section B-4.

User Note: The plastic hinging zones at the ends of SMF beams should be treated as protected zones. The plastic hinging zones should be established as part of a prequalification or qualification program for the connection, per Section 522.2.2. In general, for unreinforced connections, the protected zone will extend from the face of the column to one half of the beam depth beyond the plastic hinge point.

522.3 Panel Zone of Beam-to-Column Connections (Beam Web Parallel to Column Web)

522.3.1. Shear Strength

The required thickness of the panel zone shall be determined in accordance with the method used in proportioning the panel zone of the tested or prequalified connection. As a minimum, the required shear strength of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces.

The design shear strength shall be $\phi_v R_v$ and the allowable shear strength shall be R_v/Ω_v where

$$\phi_v = 1.0 \text{ (LRFD)} \quad \Omega_v = 1.50 \text{ (ASD)}$$

and the nominal shear strength, R_v , according to the limit state of shear yielding, is determined as specified in Specification Section 510.10.6.

522.3.2 Panel Zone Thickness

The individual thicknesses, t , of column webs and doubler plates, if used, shall conform to the following requirement:

$$t \geq (d_z + w_z)/90 \quad (522-2)$$

where

- t = thickness of column web or doubler plate, mm
- d_z = panel zone depth between continuity plates, mm
- w_z = panel zone width between column flanges, mm

Alternatively, when local buckling of the column web and doubler plate is prevented by using plug welds joining them, the total panel zone thickness shall satisfy Eq. 522-2.

522.3.3 Panel Zone Doubler Plates

Doubler plates shall be welded to the column flanges using either a complete joint-penetration groove-welded or fillet-welded joint that develops the available shear strength of the full doubler plate thickness. When doubler plates are placed against the column web, they shall be welded across the top and bottom edges to develop the proportion of the total force that is transmitted to the doubler plate. When doubler plates are placed away from the column web, they shall be placed symmetrically in pairs and welded to continuity plates to develop the proportion of the total force that is transmitted to the doubler plate.

522.4 Beam and Column Limitations

The requirements of Section 521 shall be satisfied, in addition to the following.

522.4.1 Width-Thickness Limitations

Beam and column members shall meet the requirements of Section 521.2.2, unless otherwise qualified by tests.

522.4.2 Beam Flanges

Abrupt changes in beam flange area are not permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is permitted if testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section B-1, or in a program of qualification testing in accordance with Section B-4.

522.5 Continuity Plates

Continuity plates shall be consistent with the prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section B-1, or as determined in a program of qualification testing in accordance with Section B-4.

522.6 Column-Beam Moment Ratio

The following relationship shall be satisfied at beam-to-column connections:

$$\left(\frac{\sum M_{pc}}{\sum M_{pb}} \right) > 1.0 \quad (522-3)$$

where

$\sum M_{pc}$ = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. $\sum M_{pc}$ is determined by summing the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column. It is permitted to take $\sum M_{pc}$

$$= \sum Z_c (F_{yc} - P_{uc}/A_g) (LRFD) \quad \text{or} \quad \sum Z_c [(F_{yc}/1.5) P_{ac}/A_g] (ASD),$$

as appropriate. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used

$\sum M_{pb}$ = the sum of the moments in the beams at the intersection of the beam and column centerlines. $\sum M_{pb}$ is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to take $\sum M_{pb}$ =

$$(1.1 R_y F_y b Z_b + M_{uv}) (LRFD) \quad \text{or} \quad \sum [(1.1/1.5) R_y F_y b Z_b + M_{av}] (ASD)$$

as appropriate. Alternatively, it is permitted to determine $\sum M_{pb}$ consistent with a prequalified connection design as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section B-1, or in a program of qualification testing in accordance with Section B-4. When connections with reduced beam sections are used, it is permitted to take

$$\sum M_{pb} = \sum (1.1 R_y F_y b Z_{RBS} + M_{uv}) (LRFD) \quad \text{or}$$

$$\sum [(1.1/1.5) R_y F_y b Z_{RBS} + M_{av}] (ASD) \text{ appropriate}$$

A_g = gross area of column, mm²

F_{yc} = specified minimum yield stress of column, MPa

M_{av} = the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on ASD load combinations, N-mm.

P_{ac} = required compressive strength using ASD load combinations, (a positive number) N

P_{uc} = required compressive strength using LRFD load combinations, (a positive number) N

Z_b = plastic section modulus of the beam, mm³

Z_c = plastic section modulus of the column, mm³

Z_{RBS} = minimum plastic section modulus at the reduced beam section, mm³

Exception:

This requirement does not apply if either of the following two conditions is satisfied:

1. Columns with $P_{rc} < 0.3 P_c$ for all load combinations other than those determined using the amplified seismic load that satisfy either of the following:

a. Columns used in a one-story building or the top story of a multistory building.

b. Columns where: (1) the sum of the available shear strengths of all exempted columns in the story is less than 20 percent of the sum of the available shear strengths of all moment frame columns in the story acting in the same direction; and (2) the sum of the available shear strengths of all exempted columns on each moment frame column line within that story is less than 33 percent of the available shear strength of all moment frame columns on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10 percent of the plan dimension perpendicular to the line of columns.

where

For design according to Specification Section 502.3.3 (LRFD),

$$P_c = F_{yc} A_g, N$$

$$P_{rc} = P_{uc}, \text{ required compressive strength, using LRFD load combinations, N.}$$

For design according to Specification Section 502.3.4 (ASD),

$$P_c = F_{yc} A_g / 1.5, N$$

$$P_{rc} = P_{ac}, \text{ required compressive strength, using ASD load combinations, N}$$

2. Columns in any story that has a ratio of available shear strength to required shear strength that is 50 percent greater than the story above.

522.7 Lateral Bracing at Beam-to-Column Connections

522.7.1 Braced Connections

Column flanges at beam-to-column connections require lateral bracing only at the level of the top flanges of the beams, when the webs of the beams and column are coplanar, and a column is shown to remain elastic outside of the panel zone. It shall be permitted to assume that the column remains elastic when the ratio calculated using Eq. 522-3 is greater than 2.0.

When a column cannot be shown to remain elastic outside of the panel zone, the following requirements shall apply:

The column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. Lateral bracing shall be either direct or indirect.

User Note: Direct lateral support (bracing) of the column flange is achieved through use of braces or other members, deck and slab, attached to the column flange at or near the desired bracing point to resist lateral buckling. Indirect lateral support refers to bracing that is achieved through the stiffness of members and connections that are not directly attached to the column flanges, but rather act through the column web or stiffener plates.

1. Each column-flange lateral brace shall be designed for a required strength that is equal to 2 percent of the available beam flange strength $F_y b_f t_{bf}$ (LRFD) or $F_y b_f t_{bf} / 1.5$ (ASD), as appropriate.

522.7.2 Unbraced Connections

A column containing a beam-to-column connection with no lateral bracing transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral braces as the column height for buckling transverse to the seismic frame and shall conform to Specification 508, except that:

1. The required column strength shall be determined from the appropriate load combinations in the NSCP code, except that E shall be taken as the lesser of:
 - a. The amplified seismic load.
 - b. 125 percent of the frame available strength based upon either the beam available flexural strength or panel zone available shear strength.
2. The slenderness L/r for the column shall not exceed 60.
3. The column required flexural strength transverse to the seismic frame shall include that moment caused by the

application of the beam flange force specified in Section 522.7.2 in addition to the second-order moment due to the resulting column flange displacement.

522.8 Lateral Bracing of Beams

Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = 0.06 r_y E / F_y$. Braces shall meet the provisions of Eqs. A-1-7 and A-1-8 of Appendix A-1.6 of the Specification, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_r = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$.

In addition, lateral braces shall be placed near concentrated forces, changes in cross-section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the SMF. The placement of lateral bracing shall be consistent with that documented for a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section B-1, or in a program of qualification testing in accordance with Section B-4.

The required strength of lateral bracing provided adjacent to plastic hinges shall be $P_u = 0.06 M_u / h_o$ (LRFD) or $P_a = 0.06 M_a / h_o$ (ASD), as appropriate, where h_o is the distance between flange centroids; and the required stiffness shall meet the provisions of Eq. A-1-8 of Appendix A-1.6 of the Specification.

522.9 Column Splices

Column splices shall comply with the requirements of Section 521.4.1. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds that meet the requirements of Section 520.3.2. Weld tabs shall be removed. When column splices are not made with groove welds, they shall have a required flexural strength that is at least equal to $R_y F_y Z_x$ (LRFD) or $R_y F_y Z_x / 1.5$ (ASD), as appropriate, of the smaller column. The required shear strength of column web splices shall be at least equal to $\sum M_{pc} / H$ (LRFD) or $\sum M_{pc} / 1.5 H$ (ASD), as appropriate, where $\sum M_{pc}$ is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Exception:

The required strength of the column splice considering appropriate stress concentration factors or fracture mechanics stress intensity factors need not exceed that determined by inelastic analyses

SECTION 523

INTERMEDIATE MOMENT FRAMES (IMF)

523.1 Scope

Intermediate moment frames (IMF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake. IMF shall meet the requirements in this Section.

523.2 Beam-to-Column Connections

523.2.1 Requirements

Beam-to-column connections used in the seismic load resisting system (SLRS) shall satisfy the requirements of Section 522.2, with the following exceptions:

1. The required interstory drift angle shall be a minimum of 0.02 radian.
2. The required strength in shear shall be determined as specified in Section 522.2.1, except that a lesser value of V_u or V_a , as appropriate, is permitted if justified by analysis. The required shear strength need not exceed the shear resulting from the application of appropriate load combinations in the NSCP code using the amplified seismic load.

523.2.2 Conformance Demonstration

Conformance demonstration shall be as described in Section 522.2.2 to satisfy the requirements of Section 523.2.1 for IMF, except that a connection prequalified for IMF in accordance with ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section B-1, or as determined in a program of qualification testing in accordance with Section B-4.

523.2.3 Welds

Unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Section B-1, or as determined in a program of qualification testing in accordance with Section B-4, complete joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be demand critical welds as described in Section 520.3.2.

User Note: For the designation of demand critical welds, standards such as ANSI/AISC 358 and tests addressing specific connections and joints should be used in lieu of the more general terms of these Provisions. Where these Provisions indicate that a particular weld is designated demand critical, but the more specific standard or test does not make such a designation, the more specific standard or test should govern. Likewise, these standards and tests may designate welds as demand critical that are not identified as such by these Provisions.

523.2.4 Protected Zone

The region at each end of the beam subject to inelastic straining shall be treated as a protected zone, and shall meet the requirements of Section 520.4. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section B-1, or as determined in a program of qualification testing in accordance with Section B-4.

User Note: The plastic hinging zones at the ends of IMF beams should be treated as protected zones. The plastic hinging zones should be established as part of a prequalification or qualification program for the connection. In general, for unreinforced connections, the protected zone will extend from the face of the column to one half of the beam depth beyond the plastic hinge point.

523.3 Panel Zone of Beam-to-Column Connections (Beam Web Parallel to Column Web)

No additional requirements beyond the Specification.

523.4 Beam and Column Limitations.

The requirements of Section 521.1 shall be satisfied, in addition to the following.

523.4.1 Width-Thickness Limitations

Beam and column members shall meet the requirements of Section 521.2.1, unless otherwise qualified by tests.

523.4.2 Beam Flanges

Abrupt changes in beam flange area are not permitted in plastic hinge regions. Drilling of flange holes or trimming of beam flange width is permitted if testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section B-1, or in a program of qualification testing in accordance with Section B-4.

523.5 Continuity Plates

Continuity plates shall be provided to be consistent with the prequalified connections designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section B-1, or as determined in a program of qualification testing in accordance with Section B-4.

523.6 Column-Beam Moment Ratio

No additional requirements beyond the Specification. Lateral Bracing at beam-to-column connections no additional requirements beyond the Specification. Lateral bracing of beams both flanges shall be laterally braced directly or indirectly. The unbraced length between lateral braces shall not exceed $0.17R_y E/F_y$. Braces shall meet the provisions of Eqs. A-1-7 and A-1-8 of Appendix A-1.6 of the Specification, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_u = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam, and $C_d = 1.0$.

In addition, lateral braces shall be placed near concentrated loads, changes in cross-section and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the IMF. Where the design is based upon assemblies tested in accordance with Section B-4, the placement of lateral bracing for the beams shall be consistent with that used in the tests or as required for prequalification in Section B-1. The required strength of lateral bracing provided adjacent to plastic hinges shall be $P_u = 0.06 M_u / h_o$ (LRFD) or $P_a = 0.06 M_a / h_o$ (ASD), as appropriate, where h_o = distance between flange centroids; and the required stiffness shall meet the provisions of Eq. A-1-8 of Appendix A-1.6 of the Specification. Column Splices Column splices shall comply with the requirements of Section 521.4.1. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds that meet the requirements of Section 520.3.2.

SECTION 524 ORDINARY MOMENT FRAMES (OMF)

524.1 Scope

Ordinary moment frames (OMF) are expected to withstand minimal inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake. OMF shall meet the requirements of this Section. Connections in conformance with Sections 522.2.2 and 522.5 or Sections 523.2.2 and 523.5 shall be permitted for use in OMF without meeting the requirements of Sections 524.2.1, 524.2.1, and 524.5.

User Note: While these provisions for OMF were primarily developed for use with wide flange shapes, with judgment, they may also be applied to other shapes such as channels, built-up sections, and hollow structural sections (HSS).

524.2 Beam-to-Column

Connections beam-to-column connections shall be made with welds and/or high-strength bolts. Connections are permitted to be fully restrained (FR) or partially restrained (PR) moment connections as follows.

524.2.1 Requirements for FR Moment Connections

FR moment connections that are part of the seismic load resisting system (SLRS) shall be designed for a required flexural strength that is equal to $1.1 R_y M_p$ (LRFD) or $(1.1/1.5) R_y M_p$ (ASD), as appropriate, of the beam or girder, or the maximum moment that can be developed by the system, whichever is less.

FR connections shall meet the following requirements.

1. Where steel backing is used in connections with Complete-Joint-Penetration (CJP) beam flange groove welds, steel backing and tabs shall be removed, except that top-flange backing attached to the column by a continuous fillet weld on the edge below the CJP groove weld need not be removed. Removal of steel backing and tabs shall be as follows:
 - a. Following the removal of backing, the root pass shall be back gouged to sound weld metal and back welded with a reinforcing fillet. The reinforcing fillet shall have a minimum leg size of 8 mm.

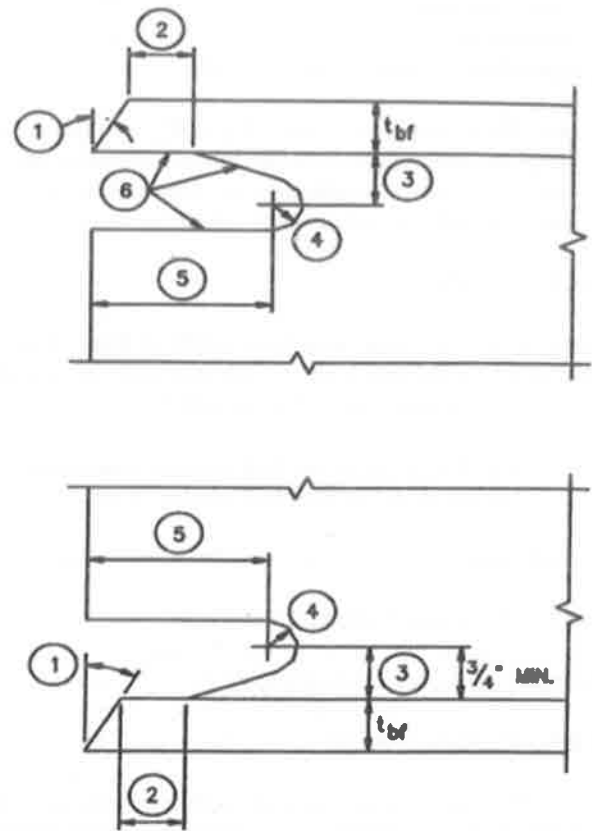
b. Weld tab removal shall extend to within 3 mm of the base metal surface, except at continuity plates where removal to within 6 mm of the plate edge is acceptable. Edges of the weld tab shall be finished to a surface roughness value of **13 μm** or better. Grinding to a flush condition is not required. Gouges and notches are not permitted. The transitional slope of any area where gouges and notches have been removed shall not exceed 1:5. Material removed by grinding that extends more than 2 mm below the surface of the base metal shall be filled with weld metal. The contour of the weld at the ends shall provide a smooth transition, free of notches and sharp corners.

- Where weld access holes are provided, they shall be as shown in Figure 524-1. The weld access hole shall have a surface roughness value not to exceed **13 μm** , and shall be free of notches and gouges. Notches and gouges shall be repaired as required by the engineer-of-record. Weld access holes are prohibited in the beam web adjacent to the end-plate in bolted moment end-plate connections.
- The required strength of double-sided partial-joint-penetration groove welds and double-sided fillet welds that resist tensile forces in connections shall be **1. $1R_y F_y A_g$ (LRFD)** or **(1.1/1.5) $R_y F_y A_g$ (ASD)**, as appropriate, of the connected element or part. Single-sided partial-joint-penetration groove welds and single-sided fillet welds shall not be used to resist tensile forces in the connections.
- For FR moment connections, the required shear strength, **V_u** or **V_a** , as appropriate, of the connection shall be determined using the following quantity for the earthquake load effect **E** :

$$E = 2[1.1 R_y M_p] / L_h \quad (524-1)$$

Where this **E** is used in ASD load combinations that are additive with other transient loads and that are based on SEI/ASCE 7, the 0.75 combination factor for transient loads shall not be applied to **E** .

Alternatively, a lesser value of **V_u** or **V_a** is permitted if justified by analysis. The required shear strength need not exceed the shear resulting from the application of appropriate load combinations in the NSCP code using the amplified seismic load.



User Note:

- Bevel as required for selected groove weld
- Larger of t_{bf} or $\frac{1}{2}$ in (plus $\frac{1}{2}t_{bf}$ or minus $\frac{1}{4}t_{bf}$)
- $\frac{3}{4}t_{bf}$ to t_{bf} , $\frac{3}{4}$ in minimum ($\pm \frac{1}{4}$ in) (± 6 mm.)
- $\frac{3}{8}$ in minimum radius (plus not limited, minus 0)
- $3t_{bf}$ ($\pm \frac{1}{2}$ in) (± 13 mm)

Tolerance shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25°

Figure 524-1 Weld-Access Hole Detail (from FEMA 350, "Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings")

524.2.2 Requirements for PR Moment Connections

PR moment connections are permitted when the following requirements are met:

- Such connections shall be designed for the required strength as specified in Section 524.2.1 above.
- The nominal flexural strength of the connection, **M_n** , shall be no less than 50 percent of **M_p** of the connected beam or column, whichever is less.

3. The stiffness and strength of the PR moment connections shall be considered in the design, including the effect on overall frame stability.
4. For PR moment connections, V_u or V_a , as appropriate, shall be determined from the load combination above plus the shear resulting from the maximum end moment that the connection is capable of resisting.

524.2.3 Welds

Complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be demand critical welds as described in Section 520.3.2.

524.3 Panel Zone of Beam-to-Column Connections (Beam Web Parallel To Column Web)

No additional requirements beyond the Specification.

524.4 Beam and Column Limitations

No requirements beyond Section 521.1.

524.5 Continuity Plates

When FR moment connections are made by means of welds of beam flanges or beam-flange connection plates directly to column flanges, continuity plates shall be provided in accordance with Section 510 of the Specification. Continuity plates shall also be required when:

$$t_{cf} < 0.54 \sqrt{b_f t_{bf} F_{yb} / F_{yc}}$$

or when

$$t_{cf} < b_f / 6$$

Where continuity plates are required, the thickness of the plates shall be determined as follows:

1. For one-sided connections, continuity plate thickness shall be at least one half of the thickness of the beam flange.
2. For two-sided connections the continuity plates shall be at least equal in thickness to the thicker of the beam flanges.

The welded joints of the continuity plates to the column flanges shall be made with either complete-joint-penetration groove welds, two-sided partial-joint penetration groove welds combined with reinforcing fillet welds, or two-sided fillet welds. The required strength of these joints shall not be less than the available strength of the contact area of the plate with the column flange. The required strength of the welded joints of the continuity plates to the column web shall be the least of the following:

- a. The sum of the available strengths at the connections of the continuity plate to the column flanges.

The available shear strength of the contact area of the plate with the column web.

- b. The weld available strength that develops the available shear strength of the column panel zone.
- c. The actual force transmitted by the stiffener.

524.6 Column-Beam Moment Ratio

No additional requirements beyond the specification

524.7 Lateral Bracing at Beam-to-Column Connections

No additional requirements beyond the Specification.

524.8 Lateral Bracing of Beams

No additional requirements beyond the Specification.

524.9 Column Splices

Column splices shall comply with the requirements of Section 521.4.1.

SECTION 525

SPECIAL TRUSS MOMENT FRAMES (STMF)

525.1 Scope

Special truss moment frames (STMF) are expected to withstand significant inelastic deformation within a specially designed segment of the truss when subjected to the forces from the motions of the design earthquake. STMF shall be limited to span lengths between columns not to exceed 20 m and overall depth not to exceed 1.8 m. The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment. STMF shall meet the requirements in this Section.

525.2 Special Segment

Each horizontal truss that is part of the seismic load resisting system (SLRS) shall have a special segment that is located between the quarter points of the span of the truss. The length of the special segment shall be between 0.1 and 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 nor be less than 0.67.

Panels within a special segment shall either be all Vierendeel panels or all X-braced panels; neither a combination thereof nor the use of other truss diagonal configurations is permitted. Where diagonal members are used in the special segment, they shall be arranged in an X pattern separated by vertical members. Such diagonal members shall be interconnected at points where they cross. The interconnection shall have a required strength equal to 0.25 times the nominal tensile strength of the diagonal member. Bolted connections shall not be used for web members within the special segment. Diagonal web members within the special segment shall be made of flat bars of identical sections.

Splicing of chord members is not permitted within the special segment, nor within one-half the panel length from the ends of the special segment. The required axial strength of the diagonal web members in the special segment due to dead and live loads within the special segment shall not exceed $0.03F_yA_g$ (LRFD) or $(0.03/1.5)F_yA_g$ (ASD), as appropriate.

The special segment shall be a protected zone meeting the requirements of Section 520.4.

525.3 Strength of Special Segment Members

The available shear strength of the special segment shall be calculated as the sum of the available shear strength of the chord members through flexure, and the shear strength corresponding to the available tensile strength and 0.3 times the available compressive strength of the diagonal members, when they are used. The top and bottom chord members in the special segment shall be made of identical sections and shall provide at least 25 percent of the required vertical shear strength. The required axial strength in the chord members, determined according to the limit state of tensile yielding, shall not exceed 0.45 times ϕP_n (LRFD) or P_n/Ω (ASD), as appropriate,

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where

$$P_n = F_y A_g$$

The end connection of diagonal web members in the special segment shall have a required strength that is at least equal to the expected yield strength, in tension, of the web member, $R_y F_y A_g$ (LRFD) or $R_y F_y A_g/1.5$ (ASD), as appropriate.

525.4 Strength of Non-Special Segment RS

Members and connections of STMF, except those in the special segment specified in Section 525.2, shall have a required strength based on the appropriate load combinations in the NSCP code, replacing the earthquake load term E with the lateral loads necessary to develop the expected vertical shear strength of the special segment V_{ne} (LRFD) or $V_{ne}/1.5$ (ASD), as appropriate, at mid-length, given as:

$$V_{ne} = \frac{3.75R_y M_{nc}}{L_s} + 0.075EI \frac{(L - L_y)}{L_c^3} + R_y(P_{nt} + 0.3P_{nc}) \sin \alpha \quad (525-1)$$

where

- M_{nc} = nominal flexural strength of a chord member of the special segment, N-mm
- EI = flexural elastic segment of a chord member of the special segment, N-mm²
- L = span length of the truss, mm
- L_s = length of the special segment, in
- P_{nt} = nominal tensile strength of a diagonal member of the special segment, kips (N)
- P_{nc} = nominal compressive strength of a diagonal member of the special segment, kips, N

α = angle of diagonal members with the horizontal

525.5 Width-Thickness Limitations

Chord members and diagonal web members within the special segment shall meet the requirements of Section 521.2.2.

525.6 Lateral Bracing

The top and bottom chords of the trusses shall be laterally braced at the ends of the special segment, and at intervals not to exceed L_p according to Specification Section 506, along the entire length of the truss. The required strength of each lateral brace at the ends of and within the special segment shall be

$$P_u = 0.06R_yP_{nc}(\text{LRFD}) \text{ or}$$

$$P_a = (0.06/1.5)R_yP_{nc}(\text{ASD}), \text{ as appropriate.}$$

where

P_{nc} = is the nominal compressive strength of the special segment chord member

Lateral braces outside of the special segment shall have a required strength of

$$P_u = 0.02R_yP_{nc}(\text{LRFD}) \text{ or}$$

$$P_a = (0.02/1.5)R_yP_{nc}(\text{ASD}), \text{ as appropriate.}$$

The required brace stiffness shall meet the provisions of Eq. A-1-4 of Appendix A-1.6 of the Specification, where

$$P_r = P_u = R_yP_{nc}(\text{LRFD}) \text{ or}$$

$$P_r = P_a = R_yP_{nc}/1.5 (\text{ASD}), \text{ as appropriate.}$$

SECTION 526

SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF)

526.1 Scope

Special concentrically braced frames (SCBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. SCBF shall meet the requirements in this Section.

User Note: Section 527 (OCBF) should be used for the design of tension-only bracing.

526.2 Members

526.2.1 Slenderness

Bracing members shall have $Kl/r \leq 4\sqrt{\frac{E}{F_y}}$

Exception:

Braces with $4\sqrt{E/F_y} < Kl/r \leq 200$ are permitted in frames in which the available strength of the column is at least equal to the maximum load transferred to the column considering R_y (LRFD) or $(1/1.5)R_y$ (ASD), as appropriate, times the nominal strengths of the connecting brace elements of the building. Column forces need not exceed those determined by inelastic analysis, nor the maximum load effects that can be developed by the system.

526.2.2 Required Strength

Where the effective net area of bracing members is less than the gross area, the required tensile strength of the brace based upon the limit state of fracture in the net section shall be greater than the lesser of the following:

1. The expected yield strength, in tension, of the bracing member, determined as $R_yF_yA_g$ (LRFD) or $R_yF_yA_g/1.5$ (ASD), as appropriate.
2. The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

User Note: This provision applies to bracing members where the section is reduced. A typical case is a slotted HSS brace at the gusset plate connection.

526.2.3 Lateral Force Distribution

Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal force along that line is resisted by braces in tension, unless the available strength of each brace in compression is larger than the required strength resulting from the application of the appropriate load combinations stipulated by the NSCP code including the amplified seismic load. For the purposes of this provision, a line of bracing is defined as a single line or parallel lines with a plan offset of 10 percent or less of the building dimension perpendicular to the line of bracing.

526.2.4 Width-Thickness Limitations

Column and brace members shall meet the requirements of Section 521.2.2.

User Note: HSS walls may be stiffened to comply with this requirement.

526.2.5 Built-up Members

The spacing of stitches shall be such that the slenderness ratio L/r of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member.

The sum of the available shear strengths of the stitches shall equal or exceed the available tensile strength of each element. The spacing of stitches shall be uniform. Not less than two stitches shall be used in a built-up member. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

Exception:

Where the buckling of braces about their critical buckling axis does not cause shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio L/r of the individual elements between the stitches does not exceed 0.75 times the governing slenderness ratio of the built-up member.

526.3 Required Strength of Bracing Connections

526.3.1 Required Tensile Strength

The required tensile strength of bracing connections (including beam-to-column connections if part of the bracing system) shall be the lesser of the following:

1. The expected yield strength, in tension, of the bracing member, determined as $R_y F_y A_g$ (LRFD) or

$R_y F_y A_g / 1.5$ (ASD), as appropriate.

2. The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

526.3.2 Required Flexural Strength

The required flexural strength of bracing connections shall be equal to $1.1 R_y M_p$ (LRFD) or $(1.1 / 1.5) R_y M_p$ (ASD), as appropriate, of the brace about the critical buckling axis.

Exception:

Braced connections that meet the requirements of Section 526.3.1 and can accommodate the inelastic rotations associated with braced post-buckling deformations need not meet this requirement.

User Note: Accommodation of inelastic rotation is typically accomplished by means of a single gusset plate with the brace terminating before the line of restraint. The detailing requirements for such a connection are described in the AISC commentary.

526.3.3 Required Compressive Strength

Bracing connections shall be designed for a required compressive strength based on buckling limit states that is at least equal to $1.1 R_y P_n$ (LRFD) or $(1.1 / 1.5) R_y P_n$ (ASD), as appropriate, where P_n is the nominal compressive strength of the brace.

526.4 Special Bracing Configuration Requirements

526.4.1 V-Type and Inverted V-Type Bracing.

V-type and inverted V-type SCBF shall meet the following requirements:

1. The required strength of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the NSCP code assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E , on the beam shall be determined as follows:
 - a. The forces in all braces in tension shall be assumed to be equal to $R_y F_y A_g$.
 - b. The forces in all adjoining braces in compression shall be assumed to be equal to $0.3 P_n$.

2. Beams shall be continuous between columns. Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$, as specified by Eq. A-1.1-7 and A-1.1-8 of Appendix A-1 of the Specification. Lateral braces shall meet the provisions of Eqs. A-1.6-7 and A-1.6-8 of Appendix A-1.6 of the Specification, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_u = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$.

As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

User Note: One method of demonstrating sufficient out-of-plane strength and stiffness of the beam is to apply the bracing force defined in Eq. A-1.6-7 of Appendix A-1.6 of the Specification to each flange so as to form a torsional couple; this loading should be in conjunction with the flexural forces defined in item (1) above. The stiffness of the beam (and its restraints) with respect to this torsional loading should be sufficient to satisfy Eq. A-1.6-8.

526.4.2 K-Type

Bracing K-type braced frames are not permitted for SCBF.

526.5 Column Splices

In addition to meeting the requirements in Section 521.4, column splices in SCBF shall be designed to develop 50 percent of the lesser available flexural strength of the connected members. The required shear strength shall be $\Sigma M_{pc}/H$ (LRFD) or $\Sigma M_{pc}/1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

526.6 Protected Zone

The protected zone of bracing members in SCBF shall include the center one-quarter of the brace length, and a zone adjacent to each connection equal to the brace depth in the plane of buckling. The protected zone of SCBF shall include elements that connect braces to beams and columns and shall satisfy the requirements of Section 520.4.

SECTION 527 ORDINARY CONCENTRICALLY BRACED FRAMES (OCBF)

527.1 Scope

Ordinary concentrically braced frames (OCBF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake. OCBF shall meet the requirements in this section. OCBF above the isolation system in seismically isolated structures shall meet the requirements of Sections 527.4 and 527.5 and need not meet the requirements of Sections 527.2 and 527.3.

527.2 Bracing Members

Bracing members shall meet the requirements of Section 521.2.2.

Exception:

HSS braces that are filled with concrete need not comply with this provision.

Bracing members in K, V, or inverted V-type configurations shall have $Kl/r \leq 4\sqrt{E/F_y}$

User Note: Bracing members that are designed as tension only (that is, neglecting their strength in compression) are not appropriate for K, V, and inverted V configurations. Such braces may be used in other configurations and are not required to satisfy this provision. Such members may include slender angles, plate, or cable bracing, which are not excluded by Section 519.1.

527.3 Special Bracing Configuration

Requirements Beams in V-type and inverted V-type OCBF and columns in K-type OCBF shall be continuous at bracing connections away from the beam-to-column connection and shall meet the following requirements:

1. The required strength shall be determined based on the load combinations of the NSCP code assuming that the braces provide no support of dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E , on the member shall be determined as follows:
 - a. The forces in braces in tension shall be assumed to be equal to $R_y F_y A_g$. For V-type and inverted V-type OCBF, the forces in braces in tension need not exceed the maximum force that can be developed by the system.

- b. The forces in braces in compression shall be assumed to be equal to $0.3P_n$.
2. Both flanges shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$, as specified by Eqs. A-1.7-7 and A-1.8-8 of Appendix A-1 of the Specification. Lateral braces shall meet the provisions of Eqs. A-1.6-7 and A-1.6-8 of Appendix A-1.6 of the Specification, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_u = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$. As a minimum, one set of lateral braces is required at the point of intersection of the bracing, unless the member has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

User Note: See User Note in Section 526.4 for a method of establishing sufficient out-of-plane strength and stiffness of the beam.

527.4 Bracing Connections

The required strength of bracing connections shall be determined as follows.

1. For the limit state of bolt slip, the required strength of bracing connections shall be that determined using the load combinations stipulated by the NSCP code, not including the amplified seismic load.
2. For other limit states, the required strength of bracing connections is the expected yield strength, in tension, of the brace, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.

Exception:

The required strength of the brace connection need not exceed either of the following:

1. *The maximum force that can be developed by the system*
2. *A load effect based upon using the amplified seismic load*

527.5 OCBF above Seismic Isolation Systems

527.5.1 Bracing Members

Bracing members shall meet the requirements of Section 521.2.2 and shall have

$$Kl/r \leq 4 \sqrt{E/F_y}$$

527.5.2 K-Type Bracing

K-type braced frames are not permitted.

527.5.3 V-Type and Inverted-V-Type Bracing.

Beams in V-type and inverted V-type bracing shall be continuous between columns.

SECTION 528

ECCENTRICALLY BRACED FRAMES (EBF)

528.1 Scope

Eccentrically braced frames (EBFs) are expected to withstand significant inelastic deformations in the links when subjected to the forces resulting from the motions of the design earthquake. The diagonal braces, columns, and beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened links, except where permitted in this Section. In buildings exceeding five stories in height, the upper story of an EBF system is permitted to be designed as an OCBF or a SCBF and still be considered to be part of an EBF system for the purposes of determining system factors in the NSCP code. EBF shall meet the requirements in this section.

528.2 Links

528.2.1 Limitations

Links shall meet the requirements of Section 521.2.2.

The web of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted.

528.2.2. Shear Strength

Except as limited below, the link design shear strength, $\phi_v V_n$, and the allowable shear strength, V_n/Ω_v , according to the limit state of shear yielding shall be determined as follows:

V_n = nominal shear strength of the link, equal to the lesser of V_p or $2M_p/e$, N

$$\phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

where

$$M_p = F_y Z, \text{ N-mm}$$

$$V_p = 0.6 F_y A_w, \text{ N}$$

$$e = \text{link length, mm}$$

$$A_w = (d - 2t_f)t_w$$

The effect of axial force on the link available shear strength need not be considered if

$$P_u \leq 0.15 P_y \text{ (LRFD) or}$$

$$P_a \leq (0.15/1.5) P_y \text{ (ASD), as appropriate.}$$

where

P_u = required axial strength using LRFD load combinations, N

P_a = required axial strength using ASD load combinations, N

P_y = nominal axial yield strength = $F_y A_g$, N

If $P_u > 0.15 P_y$ (LRFD) or

$P_a > (0.15/1.5) P_y$ (ASD), as appropriate, the following additional requirements shall be met:

1. The available shear strength of the link shall be the lesser of $\phi_v V_{pa}$ and $2\phi_v M_{pa}/e$ (LRFD) or V_{pa}/Ω_v and $2(M_{pa}/e)/\Omega_v$ (ASD), as appropriate,

where

$$\phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

$$V_{pa} = V_p \quad (528-1)$$

$$M_{pa} = 1.18 M_p \quad (528-2)$$

$P_r = P_u$ (LRFD) or

P_a (ASD), as appropriate

$P_c = P_y$ (LRFD) or $P_y/1.5$ (ASD), as appropriate

2. The length of the link shall not exceed:

$$a. \quad [1.15 - 0.5\rho'(A_w/A_g)] 1.6 M_p/V_p$$

$$\text{When } \rho'(A_w/A_g) \geq 0.3 \quad (528-3)$$

nor

$$b. \quad 1.6 M_p/V_p$$

$$\text{When } \rho'(A_w/A_g) < 0.3 \quad (528-4)$$

where

$$A_w = (d - 2t_f)t_w$$

$$\rho' = P_r/V_r$$

where

- V_r = V_u (LRFD) or V_a (ASD), as appropriate
 V_u = required shear strength based on LRFD load combinations, N
 V_a = required shear strength based on ASD load combinations, N

528.2.3 Link Rotation Angle

The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift, Δ . The link rotation angle shall not exceed the following values:

1. 0.08 radians for links of length $1.6M_p/V_p$ or less.
2. 0.02 radians for links of length $2.6M_p/V_p$ or greater.
3. The value determined by linear interpolation between the above values for links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$.

528.3 Link Stiffeners

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than $0.75t_w$ or 10 mm,

Whichever is larger, where b_f and t_w are the link flange width and link web thickness respectively.

Links shall be provided with intermediate web stiffeners as follows:

1. Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle of 0.08 radian or $(52t_w - d/5)$ for link rotation angles of 0.02 radian or less. Linear interpolation shall be used for values between 0.08 and 0.02 radian.
2. Links of length greater than $2.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at a distance of 1.5 times b_f from each end of the link.
3. Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ shall be provided with intermediate web stiffeners

meeting the requirements of (a) and (b) above.

4. Intermediate web stiffeners are not required in links of lengths greater than $5M_p/V_p$.
5. Intermediate web stiffeners shall be full depth. For links that are less than 635 mm in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or 10 mm, whichever is larger, and the width shall be not less than $(b_f/2) - t_w$. For links that are 635 mm in depth or greater, similar intermediate stiffeners are required on both sides of the web.

The required strength of fillet welds connecting a link stiffener to the link web is $A_{st}F_y$ (LRFD) or $A_{st}F_y/1.5$ (ASD), as appropriate, where A_{st} is the area of the stiffener. The required strength of fillet welds connecting the stiffener to the link flanges is $A_{st}F_y/4$ (LRFD) or $A_{st}F_y/4(1.5)$ (ASD).

528.4 Link-to-Column Connections

Link-to-column connections must be capable of sustaining the maximum link rotation angle based on the length of the link, as specified in Section 528.2.3. The strength of the connection measured at the column face shall equal at least the nominal shear strength of the link, V_n , as specified in Section 528.2.2 at the maximum link rotation angle. Link-to-column connections shall satisfy the above requirements by one of the following:

1. Use a connection prequalified for EBF in accordance with Section B-1.
2. Provide qualifying cyclic test results in accordance with Section B-4. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:
 - a. Tests reported in research literature or documented tests performed for other projects that are representative of project conditions, within the limits specified in Section B-4.
 - b. Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Section B-4.

Exception:

Where reinforcement at the beam-to-column connection at the link end precludes yielding of the beam over the reinforced length, the link is permitted to be the beam segment from the end of the reinforcement to the brace connection. Where such links are used and the link length does not exceed $1.6M_p/V_p$, cyclic testing of the reinforced connection is not required if the available strength of the reinforced section and the connection equals or exceeds the required strength calculated based upon the strain-hardened link as described in Section 528.8. Full depth stiffeners as required in Section 528.2.3 shall be placed at the link-to-reinforcement interface.

528.5 Lateral Bracing of Link

Lateral bracing shall be provided at both the top and bottom link flanges at the ends of the link. The required strength of each lateral brace at the ends of the link shall be $P_b = 0.06 M_r/h_o$, where h_o is the distance between flange centroids in mm.

For design according to Specification Section 502.3.3 (LRFD)

$$M_r = M_{u, exp} = R_y Z F_y$$

For design according to Specification Section B3.4 (ASD)

$$M_r = M_{u, exp}/1.5$$

The required brace stiffness shall meet the provisions of Eq. A-1.6-8 of the Specification, where M_r is defined above, $C_d = 1$, and L_b is the link length.

528.6 Diagonal Brace and Beam Outside of Link**528.6.1 Diagonal Brace**

The required combined axial and flexural strength of the diagonal brace shall be determined based on load combinations stipulated by the NSCP code. For load combinations including seismic effects, a load Q_1 shall be substituted for the term E , where Q_1 is defined as the axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link $R_y V_n$, where V_n is as defined in Section 528.2.2. The available strength of the diagonal brace shall comply with Specification Section 508.

Brace members shall meet the requirements of Section 521.2.1.

528.6.2 Beam Outside Link

The required combined axial and flexural strength of the beam outside of the link shall be determined based on load combinations stipulated by the NSCP code. For load combinations including seismic effects, a load Q_1 shall be substituted for the term E where Q_1 is defined as the forces generated by at least 1.1 times the expected nominal shear strength of the link, $R_y V_n$, where V_n is as defined in Section 528.2.2. The available strength of the beam outside of the link shall be determined by the Specification, multiplied by R_y .

User Note: The diagonal brace and beam segment outside of the link are intended to remain essentially elastic under the forces generated by the fully yielded and strain hardened link. Both the diagonal brace and beam segment outside of the link are typically subject to a combination of large axial force and bending moment, and therefore should be treated as beam-columns in design, where the available strength is defined by Section 508 of the Specification.

At the connection between the diagonal brace and the beam at the link end of the brace, the intersection of the brace and beam centerlines shall be at the end of the link or in the link.

528.6.3 Bracing Connections

The required strength of the diagonal brace connections, at both ends of the braces, shall be at least equal to the required strength of the diagonal brace, as defined in Section 528.6.1. The diagonal brace connections shall also satisfy the requirements of Section 526.3.3.

No part of the diagonal brace connection at the link end of the brace shall extend over the link length. If the brace is designed to resist a portion of the link end moment, then the diagonal brace connection at the link end of the brace shall be designed as a fully-restrained moment connection.

528.7 Beam-to-Column Connections

If the EBF system factors in the NSCP code require moment resisting connections away from the link, then the beam-to-column connections away from the link shall meet the requirements for beam-to-column connections for OMF specified in Sections 11.2 and 11.5.

If the EBF system factors in the NSCP code do not require moment resisting connections away from the link, then the beam-to-column connections away from the link are permitted to be designed as pinned in the plane of the web.

528.8 Required Strength of Columns

In addition to the requirements in Section 521.3, the required strength of columns shall be determined from load combinations as stipulated by the NSCP code, except that the seismic load E shall be the forces generated by 1.1 times the expected nominal shear strength of all links above the level under consideration. The expected nominal shear strength of a link is $R_y V_n$, where V_n is as defined in Section 528.2.2.

Column members shall meet the requirements of Section 521.2.2.

528.9 Protected Zone

Links in EBFs are a protected zone, and shall satisfy the requirements of Section 520.4. Welding on links is permitted for attachment of link stiffeners, as required in Section 528.3.

528.10 Demand Critical Welds

Complete-joint-penetration groove welds attaching the link flanges and the link web to the column are demand critical welds, and shall satisfy the requirements of Section 520.3.2.

SECTION 529

BUCKLING-RESTRAINED BRACED FRAMES (BRBF)

529.1 Scope

Buckling-restrained braced frames (BRBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. BRBF shall meet the requirements in this Section. Where the NSCP code does not contain design coefficients for BRBF, the provisions of Section B-3 shall apply.

529.2 Bracing Members

Bracing members shall be composed of a structural steel core and a system that restrains the steel core from buckling.

529.2.1 Steel Core

The steel core shall be designed to resist the entire axial force in the brace.

The brace design axial strength, ϕP_{ysc} (LRFD), and the brace allowable axial strength, P_{ysc} / Ω (ASD), in tension and compression, according to the limit state of yielding, shall be determined as follows:

$$P_{ysc} = F_{ysc} A_{sc} \quad (529-1)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where

F_{ysc} = specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, MPa

A_{sc} = net area of steel core, mm²

Plates used in the steel core that are 50 mm thick or greater shall satisfy the minimum notch toughness requirements of Section 519.3.

Splices in the steel core are not permitted.

529.2.2 Buckling-Restraining System

The buckling-restraining system shall consist of the casing for the steel core. In stability calculations, beams, columns, and gussets connecting the core shall be considered parts of this system.

The buckling-restraining system shall limit local and overall buckling of the steel core for deformations corresponding to 2.0 times the design story drift. The buckling-restraining system shall not be permitted to buckle within deformations corresponding to 2.0 times the design story drift.

User Note: Conformance to this provision is demonstrated by means of testing as described in Section 529.2.3.

529.2.3 Testing

The design of braces shall be based upon results from qualifying cyclic tests in accordance with the procedures and acceptance criteria of Section B-5. Qualifying test results shall consist of at least two successful cyclic tests: one is required to be a test of a brace subassembly that includes brace connection rotational demands complying with Section B-5, Section B-5.4 and the other shall be either a uniaxial or a subassembly test complying with Section B-5, Section B-5.5. Both test types are permitted to be based upon one of the following:

1. Tests reported in research or documented tests performed for other projects.
2. Tests that are conducted specifically for the project.

Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains consistent with or less severe than the tested assemblies and that considers the adverse effects of variations in material properties. Extrapolation of test results shall be based upon similar combinations of steel core and buckling-restraining system sizes. Tests shall be permitted to qualify a design when the provisions of Section B-5 are met.

529.2.4 Adjusted Brace Strength

Where required by these Provisions, bracing connections and adjoining members shall be designed to resist forces calculated based on the adjusted brace strength.

The adjusted brace strength in compression shall be $\beta_{\omega} R_y P_{ysc}$. The adjusted brace strength in tension shall be $\omega R_y P_{ysc}$.

Exception:

The factor R_y need not be applied if P_{ysc} is established using yield stress determined from a coupon test.

The compression strength adjustment factor, β , shall be calculated as the ratio of the maximum compression force to the maximum tension force of the test specimen measured from the qualification tests specified in Section B-5, Section B-5.6.3 for the range of deformations corresponding to 2.0 times the design story drift. The larger value of β from the two required brace qualification tests shall be used. In no case shall β be taken as less than 1.0.

The strain hardening adjustment factor, ω , shall be calculated as the ratio of the maximum tension force measured from the qualification tests specified in Section B-5, Section B-5.6.3 (for the range of deformations corresponding to 2.0 times the design story drift) to F_{ysc} of the test specimen. The larger value of ω from the two required qualification tests shall be used. Where the tested steel core material does not match that of the prototype, ω shall be based on coupon testing of the prototype material.

529.3 Bracing Connections

529.3.1 Required Strength

The required strength of bracing connections in tension and compression (including beam-to-column connections if part of the bracing system) shall be 1.1 times the adjusted brace strength in compression (LRFD) or 1.1/1.5 times the adjusted brace strength in compression (ASD).

529.3.2 Gusset Plates

The design of connections shall include considerations of local and overall buckling. Bracing consistent with that used in the tests upon which the design is based is required.

User Note: This provision may be met by designing the gusset plate for a transverse force consistent with transverse bracing forces determined from testing, by adding a stiffener to it to resist this force, or by providing a brace to the gusset plate or to the brace itself. Where the supporting tests did not include transverse bracing, no such bracing is required. Any attachment of bracing to the steel core must be included in the qualification testing.

529.4 Special Requirements

Related to Bracing Configuration. V-type and inverted-V-type braced frames shall meet the following requirements:

1. The required strength of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the NSCP code assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the vertical and horizontal earthquake effect, E , on the beam shall be determined from the adjusted brace strengths in tension and compression.

Beams shall be continuous between columns. Both flanges of beams shall be laterally braced. Lateral braces shall meet the provisions of Eqs. A-1.6-7 and A-1.6-8 of Appendix A-1.6 of the Specification, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_u = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$. As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

User Note: The beam has sufficient out-of-plane strength and stiffness if the beam bent in the horizontal plane meets the required brace strength and required brace stiffness for column nodal bracing as prescribed in the Specification. P_u may be taken as the required compressive strength of the brace.

For purposes of brace design and testing, the calculated maximum deformation of braces shall be increased by including the effect of the vertical deflection of the beam under the loading defined in Section 529.4.

K-type braced frames are not permitted for BRBF.

529.5 Beams and Columns

Beams and columns in BRBF shall meet the following requirements.

529.5.1 Width-Thickness Limitations

Beam and column members shall meet the requirements of Section 521.2.2.

529.5.2 Required Strength

The required strength of beams and columns in BRBF shall be determined from load combinations as stipulated in the NSCP code. For load combinations that include earthquake effects, the earthquake effect, E , shall be determined from the adjusted brace strengths in tension and compression.

The required strength of beams and columns need not exceed the maximum force that can be developed by the system.

User Note: Load effects calculated based on adjusted brace strengths should not be amplified by the overstrength factor, Ω_0 .

529.5.3 Splices

In addition to meeting the requirements in Section 521.4, column splices in BRBF shall be designed to develop 50 percent of the lesser available flexural strength of the connected members, determined based on the limit state of yielding. The required shear strength shall be $\Sigma M_{pc}/H$ (LRFD) or

$\Sigma M_{pc}/1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

529.6 Protected Zone

The protected zone shall include the steel core of bracing members and elements that connect the steel core to beams and columns, and shall satisfy the requirements of Section 520.4.

SECTION 530

SPECIAL PLATE SHEAR WALLS (SPSW)

530.1 Scope

Special plate shear walls (SPSW) are expected to withstand significant inelastic deformations in the webs when subjected to the forces resulting from the motions of the design earthquake. The horizontal boundary elements (HBEs) and vertical boundary elements (VBEs) adjacent to the webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded webs, except that plastic hinging at the ends of HBEs is permitted. SPSW shall meet the requirements of this Section. Where the NSCP code does not contain design coefficients for SPSW, the provisions of Section B-3 shall apply.

530.2 Webs

530.2.1 Shear Strength

The panel design shear strength, ϕV_n (LRFD), and the allowable shear strength, V_n/Ω (ASD), according to the limit state of shear yielding, shall be determined as follows:

$$V_n = 0.42 F_y t_w L_{cf} \sin \alpha \quad (530-1)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where

t_w = thickness of the web, mm

L_{cf} = clear distance between VBE flanges, mm

α is the angle of web yielding in radians, as measured relative to the vertical, and it is given by:

$$\tan^4 = \frac{1 + \frac{t_w L}{2A_c}}{1 + t_w h \left(\frac{1}{A_b} + \frac{h^3}{360 I_c L} \right)} \quad (530-2)$$

h = distance between HBE centerlines, mm

A_b = cross-sectional area of a HBE, mm²

A_c = cross-sectional area of a VBE, mm²

I_c = moment of inertia of a VBE taken perpendicular to the direction of the web plate line, mm⁴

L = distance between VBE centerlines, mm

530.2.2 Panel Aspect Ratio

The ratio of panel length to height, L/h , shall be limited to $0.8 < L/h \leq 2.5$.

530.2.3 Openings in Webs

Openings in webs shall be bounded on all sides by HBE and VBE extending the full width and height of the panel, respectively, unless otherwise justified by testing and analysis.

530.3 Connections of Webs to Boundary Elements

The required strength of web connections to the surrounding HBE and VBE shall equal the expected yield strength, in tension, of the web calculated at an angle α , defined by Eq. 530-2.

530.4 Horizontal and Vertical Boundary Elements

530.4.1 Required Strength

In addition to the requirements of Section 521.3, the required strength of VBE shall be based upon the forces corresponding to the expected yield strength, in tension, of the web calculated at an angle α .

The required strength of HBE shall be the greater of the forces corresponding to the expected yield strength, in tension, of the web calculated at an angle α or that determined from the load combinations in the NSCP code assuming the web provides no support for gravity loads.

The beam-column moment ratio provisions in Section 522.6 shall be met for all HBE/VBE intersections without consideration of the effects of the webs.

530.4.2 HBE-to-VBE Connections

HBE-to-VBE connections shall satisfy the requirements of Section 524.2. The required shear strength, V_u , of a HBE-to-VBE connection shall be determined in accordance with the provisions of Section 524.2, except that the required shear strength shall not be less than the shear corresponding to moments at each end equal to $1.1 R_y M_p$ (LRFD) or $(1.1/1.5) R_y M_p$ (ASD), as appropriate, together with the shear resulting from the expected yield strength in tension of the webs yielding at an angle α .

530.4.3 Width-Thickness Limitations

HBE and VBE members shall meet the requirements of Section 521.2.2.

530.4.4 Lateral Bracing

HBE shall be laterally braced at all intersections with VBE and at a spacing not to exceed $0.086 R_y E / F_y$. Both flanges of HBE shall be braced either directly or indirectly. The required strength of lateral bracing shall be at least 2 percent of the HBE flange nominal strength, $F_y b_f t_f$. The required stiffness of all lateral bracing shall be determined in accordance with Eq. A-1.6-8 of Appendix A-1.6 of the Specification. In these equations, M_r shall be computed as $R_y Z F_y$ (LRFD) or M_r shall be computed as $R_y Z F_y / 1.5$ (ASD), as appropriate, and $C_d = 1.0$. **530.4.5. VBE Splices.** VBE splices shall comply with the requirements of Section 521.4.

530.4.5 Panel Zones

The VBE panel zone next to the top and base HBE of the SPSW shall comply with the requirements in Section 522.3.

530.4.6 Stiffness of Vertical Boundary Elements

The VBE shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_c , not less than $0.00307 t_w h_4 / L$.

SECTION 531 QUALITY ASSURANCE PLAN

531.1 Scope

When required by the NSCP code or the engineer-of-record, a quality assurance plan shall be provided. The quality assurance plan shall include the requirements of Section B-2.

User Note: The quality assurance plan in Section B-2 is considered adequate and effective for most seismic load resisting systems and is strongly encouraged for use without modification. While the NSCP code requires use of a quality assurance plan based on the seismic design category, use of the quality assurance plan for any seismic load resisting system with an R greater than 3 is strongly encouraged independent of the seismic design category. Use of a response modification factor of 3 or more indicates an assumption of system, element, and connection ductility to reduce design forces. The quality assurance plan is intended to ensure that the seismic load resisting system is significantly free of defects that would greatly reduce the ductility of the system. There may be cases (for example, non-redundant major transfer members, or where work is performed in a location that is difficult to access) where supplemental testing might be advisable. Additionally, where the contractor's quality control program has demonstrated the capability to perform some tasks this plan has assigned to quality assurance, modification of the plan could be considered.

APPENDICES B-1. PREQUALIFICATION OF BEAM- COLUMN AND LINK-TO-COLUMN CONNECTIONS

Appendix B-1.1 Scope

This appendix contains minimum requirements for prequalification of beam to-column moment connections in special moment frames (SMF), intermediate moment frames (IMF), and link-to-column connections in eccentrically braced frames (EBF). Prequalified connections are permitted to be used, within the applicable limits of prequalification, without the need for further qualifying cyclic tests. When the limits of prequalification or design requirements for prequalified connections conflict with the requirements of these Provisions, the limits of prequalification and design requirements for prequalified connections shall govern.

B-1.2 General Requirements

B-1.2.1 Basis for Prequalification

Connections shall be prequalified based on test data satisfying Section B-1.3, supported by analytical studies and design models. The combined body of evidence for prequalification must be sufficient to assure that the connection can supply the required interstory drift angle for SMF and IMF systems, or the required link rotation angle for EBF, on a consistent and reliable basis within the specified limits of prequalification. All applicable limit states for the connection that affect the stiffness, strength and deformation capacity of the connection and the Seismic Load Resisting System (SLRS) must be identified. These include fracture related limit states, stability related limit states, and all other limit states pertinent for the connection under consideration. The effect of design variables listed in Section B-1.4 shall be addressed for connection prequalification.

B-1.2.2 Authority for Prequalification

Prequalification of a connection and the associated limits of prequalification shall be established by a Connection Prequalification Review Panel (CPRP) approved by the authority having jurisdiction.

Section B-1.3 Testing Requirements

Data used to support connection prequalification shall be based on tests conducted in accordance with Section B-4. The CPRP shall determine the number of tests and the variables considered by the tests for connection prequalification.

The CPRP shall also provide the same information when limits are to be changed the connection has the ability and reliability to undergo the required interstory drift angle for SMF and IMF and the required link rotation angle for EBF, where the link is adjacent to columns. The limits on member sizes for prequalification shall not exceed the limits specified in Section B-4, Section B-2.5.2.

B-1.4 Prequalification Variables

In order to be prequalified, the effect of the following variables on connection performance shall be considered. Limits on the permissible values for each variable shall be established by the CPRP for the prequalified connection.

1. Beam or link parameters:
 - a. Cross-section shape: wide flange, box, or other
 - b. Cross-section fabrication method: (rolled shape, welded shape, or other)
 - c. Depth
 - d. Weight per foot
 - e. Flange thickness
 - f. Material specification
 - g. Span-to-depth ratio (for SMF or IMF), or link length (for EBF)
 - h. Width thickness ratio of cross-section elements
 - i. Lateral bracing
 - j. Other parameters pertinent to the specific connection under consideration
2. Column parameters:
 - a. Cross-section shape: wide flange, box, or other
 - b. Cross-section fabrication method: (rolled shape, welded shape, or other)
 - c. Column orientation with respect to beam or link: (beam or link is connected to column flange, beam or link is connected to column web, beams or links are connected to both the column flange and web, or other)
 - d. Depth

- e. Weight per foot
 - f. Flange thickness
 - g. Material specification
 - h. Width-thickness ratio of cross-section elements
 - i. Lateral bracing
 - j. Other parameters pertinent to the specific connection under consideration
3. Beam (or link) column relations:
- a. Panel zone strength
 - b. Doubler plate attachment details
 - c. Column-beam (or link) moment ratio
4. Continuity plates:
- a. Identification of conditions under which continuity plates are required
 - b. Thickness, width and depth
 - c. Attachment details
5. Welds:
- a. Location, extent (including returns), type (CJP, PJP, fillet, etc.) and any reinforcement or contouring required
 - b. Filler metal classification strength and notch toughness
 - c. Details and treatment of weld backing and weld tabs
 - d. Weld access holes: size, geometry and finish
 - e. Welding quality control and quality assurance beyond that described in Section 18, including the Non Destructive Testing (NDT) method, inspection frequency, acceptance criteria and documentation requirements
6. Bolts:
- a. Bolt diameter
 - b. Bolt grade: ASTM A325, A490, or other
 - c. Installation requirements: pretensioned, snug-tight, or other
 - d. Hole type: standard, oversize, short-slot, long-slot, or other
 - e. Hole fabrication method: drilling, punching, sub-punching and reaming, or other
 - f. Other parameters pertinent to the specific connection under consideration
7. Workmanship: All workmanship parameters that exceed AISC, RCSC and AWS requirements, pertinent to the specific connection under consideration, such as:
- a. Surface roughness of thermal cut or ground edges
 - b. Cutting tolerances
 - c. Weld reinforcement or contouring
 - d. Presence of holes, fasteners or welds for attachments
8. Additional connection details: All variables pertinent to the specific connection under consideration, as established by the CPRP

B-1.5 Design Procedure

A comprehensive design procedure must be available for a prequalified connection. The design procedure must address all applicable limit states within the limits of prequalification.

B-1.6 Prequalification Record

A prequalified connection shall be provided with a written prequalification record with the following information:

- 1. General description of the prequalified connection and drawings that clearly identify key features and components of the connection
- 2. Description of the expected behavior of the connection in the elastic and inelastic ranges of behavior, intended location(s) of inelastic action, and a description of limit states controlling the strength and deformation capacity of the connection
- 3. Listing of systems for which connection is prequalified: SMF, IMF, or EBF
- 4. Listing of limits for all prequalification variables listed in Section B-1.4.
- 5. Listing of demand critical welds
- 6. Definition of the region of the connection that comprises the protected zone
- 7. Detailed description of the design procedure for the connection, as required in Section B-1.5.
- 8. List of references of test reports, research reports and other publications that provided the basis for prequalification
- 9. Summary of quality control and quality assurance procedures

B-2. QUALITY ASSURANCE PLAN

B-2.1 Scope

Quality Control (QC) and Quality Assurance (QA) shall be provided as specified in this Section.

Visual welding inspection and Non Destructive Testing (NDT) shall be conducted in accordance with a written practice by personnel qualified in accordance with Section B-6.

User Note: Section B-6, Section B-6.3 contains items to be considered in determining the qualification requirements for welding inspectors and NDT technicians.

B-2.2 Inspection and Nondestructive Testing Personnel

Bolting inspection shall be conducted in accordance with a written practice by qualified personnel.

B-2.3 Contractor Documents

The following documents shall be submitted for review by the engineer-of-record or designee, prior to fabrication or erection, as applicable:

1. Shop drawings
2. Erection drawings
3. Welding Procedure Specifications (WPS), which shall specify all applicable essential variables of AWS D1.1 and the following, as applicable
 - a. power source (constant current or constant voltage)
 - b. for demand critical welds, electrode manufacturer and trade name
4. Copies of the manufacturer's typical certificate of conformance for all electrodes, fluxes and shielding gasses to be used. Certificates of conformance shall satisfy the applicable AWS A5 requirements.
5. For demand critical welds, applicable manufacturer's certifications that the filler metal meets the supplemental notch toughness requirements, as applicable. Should the filler metal manufacturer not supply such supplemental certifications, the contractor shall have the necessary testing performed and provide the applicable test reports.
6. Manufacturer's product data sheets or catalog data for SMAW, FCAW and GMAW composite (cored) filler metals to be used. The data sheets shall describe the product, limitations of use, recommended or typical welding parameters, and storage and exposure requirements, including baking, if applicable.

The following documents shall be available for review by the engineer-of-record or designee prior to fabrication or erection, as applicable, unless specified to be submitted:

1. Material test reports for structural steel, bolts, shear connectors, and welding materials
2. Inspection procedures
3. Nonconformance procedure
4. Material control procedure
5. Bolt installation procedure
6. Welder Performance Qualification Records (WPQR), including any supplemental testing requirements
7. QC Inspector qualifications

B-2.4 Quality Assurance Agency Documents

The agency responsible for quality assurance shall submit the following documents to the authority having jurisdiction, the engineer-of-record, and the owner or owner's designee:

1. QA agency's written practices for the monitoring and control of the agency's operations. The written practice shall include:
 - a. The agency's procedures for the selection and administration of inspection personnel, describing the training, experience and examination requirements for qualification and certification of inspection personnel, and
 - b. The agency's inspection procedures, including general inspection, material controls, and visual welding inspection
2. Qualifications of management and QA personnel designated for the project
3. Qualification records for Inspectors and NDT technicians designated for the project
4. NDT procedures and equipment calibration records for NDT to be performed and equipment to be used for the project
5. Daily or weekly inspection reports
6. Non-conformance reports

B-2.5 Inspection Points and Frequencies

Inspection points and frequencies of Quality Control (QC) and Quality Assurance (QA) tasks and documentation for the Seismic Load Resisting System (SLRS) shall be as provided in the following tables.

The following entries are used in the tables:

Observe (O) - The inspector shall observe these functions on a random, daily basis. Welding operations need not be delayed pending observations.

Perform (P) - These inspections shall be performed prior to the final acceptance of the item. Where a task is noted to be performed by both QC and QA, it shall be permitted to coordinate the inspection function between QC and QA so that the inspection functions need be performed by only one party. Where QA is to rely upon inspection functions performed by QC, the approval of the engineer-of-record and the authority having jurisdiction is required.

Document (D) - The inspector shall prepare reports indicating that the work has been performed in accordance with the contract documents. The report need not provide detailed measurements for joint fit-up, WPS settings, completed welds, or other individual items listed in the Tables in Sections B-2.5.1, B-2.5.3, or B-2.5.4. For shop fabrication, the report shall indicate the piece mark of the piece inspected. For field work, the report shall indicate the reference grid lines and floor or elevation inspected. Work not in compliance with the contract documents and whether the noncompliance has been satisfactorily repaired shall be noted in the inspection report.

B-2.5.1 Visual Welding Inspection

Visual inspection of welding shall be the primary method used to confirm that the procedures, materials, and workmanship incorporated in construction are those that have been specified and approved for the project. As a minimum, tasks shall be as follows:

B-2.5.2 Nondestructive Testing (NDT) of Welds

Nondestructive testing of welds shall be performed by quality assurance personnel.

1. Procedures

Ultrasonic testing shall be performed by QA according to the procedures prescribed in Section B-6, Section B-6.1.

Magnetic particle testing shall be performed by QA according to the procedures prescribed in Section B-6, Section B-6.2.

2. Required NDT

a. k-Area NDT

When welding of doubler plates, continuity plates, or stiffeners has been performed in the k-area, the web shall be tested for cracks using Magnetic Particle Testing (MT). The MT inspection area shall include the k-area base metal within 75 mm of the weld.

b. CJP Groove Weld NDT

Ultrasonic testing shall be performed on 100 percent of CJP groove welds in materials 8 mm thick or greater. Ultrasonic testing in materials less than 8 mm thick is not required. Magnetic particle testing shall be performed on 25 percent of all beam-to-column CJP groove welds.

c. Base Metal NDT for Lamellar Tearing and Laminations

After joint completion, base metal thicker than 38 mm loaded in tension in the through thickness direction in tee and corner joints, where the connected material is greater than 19 mm and contains CJP groove welds, shall be ultrasonically tested for discontinuities behind and adjacent to the fusion line of such welds. Any base metal discontinuities found within $t/4$ of the steel surface shall be accepted or rejected on the basis of criteria of AWS D1.1 Table 6.2, where t is the thickness of the part subjected to the through-thickness strain.

d. Beam Cope and Access Hole NDT

At welded splices and connections, thermally cut surfaces of beam copes and access holes shall be tested using magnetic particle testing or penetrant testing, when the flange thickness exceeds 38 mm for rolled shapes, or when the web thickness exceeds 38 mm for built-up shapes.

e. Reduced Beam Section Repair NDT

Magnetic particle testing shall be performed on any weld and adjacent area of the reduced beam section (RBS) plastic hinge region that has been repaired by welding, or on the base metal of the RBS plastic hinge region if a sharp notch has been removed by grinding.

f. Weld Tab Removal Sites

Magnetic particle testing shall be performed on the end of welds from which the weld tabs have been removed, except for continuity plate weld tabs.

g. Reduction of Percentage of Ultrasonic Testing

The amount of ultrasonic testing is permitted to be reduced if approved by the engineer-of-record and the authority having jurisdiction. The nondestructive testing rate for an individual welder or welding operator may be reduced to 25 percent, provided the reject rate is demonstrated to be 5 percent or less of the welds tested for the welder or welding operator. A sampling of at least 40 completed welds for a job shall be made for such reduction evaluation. Reject rate is the number of welds containing rejectable defects divided by the number of welds completed. For evaluating the reject rate of continuous welds over 1 m in length where the effective throat thickness is 25 mm or less, each 300 mm increment or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 1 m in length where the effective throat thickness is greater than 25 mm, each 150 mm of length or fraction thereof shall be considered one weld.

h. Reduction of Percentage of Magnetic Particle Testing

The amount of MT on CJP groove welds is permitted to be reduced if approved by the engineer-of-record and the authority having jurisdiction. The MT rate for an individual welder or welding operator may be reduced to 10 percent, provided the reject rate is demonstrated to be 5 percent or less of the welds tested for the welder or welding operator. A sampling of at least 20 completed welds for a job shall be made for such reduction evaluation. Reject rate is the number of welds containing rejectable defects divided by the number of welds completed. This reduction is not permitted on welds in the k-area, at repair sites, weld tab and backing removal sites and access holes.

Visual Inspection Task	QC		QA	
	Task	Doc.	Task	Doc.
Before Welding	O	-	O	-
Material identification (Type/Grade)	O	-	O	-
Fit-up of Groove Welds (including joint geometry)	P/O**	-	O	-
- Joint preparation				
- Dimensions (alignment, root opening, root face, bevel)				
- Cleanliness (condition of steel surfaces)				
- Tacking (tack weld quality and location)	P/O**	-	O	-
- Backing type and fit (if applicable)				
Configuration and finish of access holes	O	-	O	-
Fit-up of Fillet Welds	P/O**	-	O	-
- Dimensions 9 alignment, gaps at root)				
- Cleanliness 9 condition of steel surfaces)				
- Tacking (tack weld quality and location)				
**Following performance of this inspection task for ten welds to be made by a given welder, with the welder demonstrating adequate understanding of requirements and possession of skills and tools to verify these items, the Perform designation of this task shall be reduce to Observe, and the welder shall perform this task, the task shall be returned to Perform until such time as the Inspector has reestablished adequate assurance that the welder will perform the inspection tasks listed.				

Visual Inspection Task During Welding	QC		QA	
	Task	Doc.	Task	Doc.
WPS followed	O	-	O	-
- Setting on welding equipment				
- Travel speed				
- Selected welding materials				
- Shielding gas type/flow rate				
- Preheat applied				
- Interpass temperature maintained (min./max.)				
- Proper position (F, V,H, OH)				
- Intermix of filler metals avoided unless approved				
Use of qualified welders	O	-	O	-
Control and handling of welding consumables	O	-	O	-
- Packaging				
- Exposure control	O	-	O	-
Environmental conditions				
- Wind speed within limits				
- Precipitation and temperature	O	-	O	-
Welding techniques				
- Interpass and final cleaning	O	-	O	-
- Each pass within profile limitations	O	-	O	-
- Each pass meets quality requirements				
No welding over cracked tacks	O	-	O	-

Visual Inspection Task After Welding	QC		QA	
	Task	Doc.	Task	Doc.
Welds cleaned	O	-	O	-
Welder identification legible	O	-	O	-
Verify size, length, and location of welds	O	-	O	-
Visually inspect welds to acceptance criteria	P	D	P	D
- Crack porhibition				
- Weld/base-metal fusion				
- Crater cross-section				
- Weld profiles				
- Weld size				
- Undercut	P	D	P	D
- Porosity				
Placement of reinforcement fillets	P	D	P	D
Backing bars removed and weld tabs removed and finished (if required)	P	D	P	D
Repair activities	P	-	P	D

Inspection Task Prior to Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Proper bolts selected for the joint detail	O	-	O	-
Proper bolting procedure selected for joint detail	O	-	O	-
Connecting elements are fabricated properly, including the appropriate faying surface condition and hole preparation, if specified, meets applicable requirements	O	-	O	-
Pre-installation verification testing conducted for fastener assemblies and method used	P	D	O	D
Proper storage provided for bolts, nuts, washers, and other fastener components	O	-	O	-

Inspection Task During Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Fastener assemblies placed in all holes and washers (if required) are properly positioned	O	-	O	-
Joint brought to the snug tight condition prior to the pretensioning operation	O	-	O	-
Fastener components not returns by the wrench prevented from rotating	O	-	O	-
Bolts are pretensioned progress systematically from most rigid point toward free edges	O	-	O	-

Inspection Task After Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Document accepted and rejected connections	P	D	P	D

3. Documentation

All NDT performed shall be documented. For shop fabrication, the NDT report shall identify the tested weld by piece mark and location in the piece. For field work, the NDT report shall identify the tested weld by location in the structure, piece mark, and location in the piece.

B-2.5.3 Inspection of Bolting

Observation of bolting operations shall be the primary method used to confirm that the procedures, materials, and workmanship incorporated in construction are those that have been specified and approved for the project. As a minimum, the tasks shall be as follows:

B-2.5.4 Other Inspections

Where applicable, the following inspection shall be performed:

Other Inspection Task	QC		QA	
	Task	Doc.	Task	Doc.
Reduce beam section (RBS) requirements, if applicable	P	D	P	D
- contour and finish				
- dimensional tolerances				
Protected zone no holes and unapproved attachments made by contractor	P	D	P	D

B-3. SEISMIC DESIGN COEFFICIENTS AND APPROXIMATE PERIOD PARAMETERS

B-3.1 Scope

This appendix contains design coefficients, system limitations and design parameters for Seismic Load Resisting Systems (SLRS) that are included in these Provisions but not yet defined in this code for Buckling-Restrained Braced Frames (BRBF) and Special Plate Shear Walls (SPSW). The values presented in Tables B-3-1 and B-3-2 in this appendix shall only be used where neither the NSCP code nor SEI/ASCE 7 contain such values.

User Note: The design coefficients and parameters presented in this appendix are taken from the 2003 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures. This appendix will be deleted from these Provisions once SEI/ASCE 7 and this codes add the BRBF and SPSW to their list of acceptable structural systems. It is expected that such parameters will be included in an appendix to SEI/ASCE 7 which is expected to be published in mid to late 2005.

B-3.2 Symbols

The following symbols are used in this appendix.

- C_d = Deflection amplification factor
- C_r, x = Parameters used for determining the approximate fundamental period
- Ω_o = System overstrength factor
- R = Response modification coefficient

Table B-3.1 Design Coefficients and Factors for Basic Seismic Load Resisting Systems

Basic Seismic Load Resisting System	Response Modification Coefficient R	System Over Strength Factor Ω_o	Deflection Amplification Factor C_d	Height Limit			
				Seismic Design Category			
				B & C (Zone 2)	D	E (Zone 4)	F (Zone 4)
Building Frame Systems							
Buckling-Restrained Braced Frames, non-moment-resisting beam-column connections	7	2	5 1/2	NL	160	160	160
Special Plate Shear Walls	7	2	6	NL	160	160	100
Buckling-Restrained Braced Frames, moment-resisting beam-column connections	8	2 1/2	5	NL	160	160	100
Dual Systems with Special Moment Frames Capable of Resisting at Least 25% of the Prescribed Seismic Forces							
Buckling-Restrained Braced Frame	8	2 1/2	5	NL	NL	NL	NL
Special Plate Shear Walls	8	2 1/2	6 1/2	NL	NL	NL	NL
NL=Not Limited)							

B-4. QUALIFYING CYCLIC TESTS OF BEAM-TO-COLUMN AND LINK-TO-COLUMN CONNECTIONS

B-4.1 Scope

This appendix includes requirements for qualifying cyclic tests of beam-to-column moment connections in special and intermediate moment frames and link-to-column connections in eccentrically braced frames, when required in these Provisions. The purpose of the testing described in this appendix is to provide evidence that a beam-to-column connection or a link-to-column connection satisfies the requirements for strength and interstory drift angle or link rotation angle in these Provisions. Alternative testing requirements are permitted when approved by the Engineer-of-Record and the authority having jurisdiction.

This appendix provides minimum recommendations for simplified test conditions.

Table B-4.1 Values of Approximate Period Parameters C_r and x

Structure Type	C_r	x
Buckling-Restrained Braced Frames	0.03	0.75
Special Plate Shear Walls	0.02	0.75

Values of Approximate Period Parameters from (AISC)

User Note: The values in this table are intended to be used in the same ways as those in Table 9.5.2.2 of SEI/ASCE 7.

B-4.2 Symbols

The numbers in parentheses after the definition of a symbol refers to the Section number in which the symbol is first used.

θ Interstory drift angle (Section B-4.6)

γ total Total link rotation angle (Section B-4.6)

B-4.3 Definitions

Complete loading cycle. A cycle of rotation taken from zero force to zero force, including one positive and one negative peak.

Interstory drift angle. Interstory displacement divided by story height, radians.

Inelastic rotation. The permanent or plastic portion of the rotation angle between a beam and the column or between a link and the column of the test specimen, measured in radians. The inelastic rotation shall be computed based on an analysis of test specimen deformations. Sources of inelastic rotation include yielding of members, yielding of connection elements and connectors, and slip between members and connection elements. For beam-to-column moment connections in special and intermediate moment frames, inelastic rotation is computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of the centerline of the beam with the centerline of the column. For link-to-column connections in eccentrically braced frames, inelastic rotation shall be computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of the centerline of the link with the face of the column.

Prototype. The connections, member sizes, steel properties, and other design, detailing, and construction features to be used in the actual building frame.

Test specimen. A portion of a frame used for laboratory testing, intended to model the prototype.

Test setup. The supporting fixtures, loading equipment, and lateral bracing used to support and load the test specimen.

Test subassembly. The combination of the test specimen and pertinent portions of the test setup.

Total link rotation angle. The relative displacement of one end of the link with respect to the other end (measured transverse to the longitudinal axis of the undeformed link), divided by the link length. The total link rotation angle shall include both elastic and inelastic components of deformation of the link and the members attached to the link ends.

B-4.4 Test Subassembly Requirements

The test subassembly shall replicate as closely as is practical the conditions that will occur in the prototype during earthquake loading. The test subassembly shall include the following features:

1. The test specimen shall consist of at least a single column with beams or links attached to one or both sides of the column.
2. Points of inflection in the test assembly shall coincide approximately with the anticipated points of inflection in the Prototype under earthquake loading.

3. Lateral bracing of the test subassembly is permitted near load application or reaction points as needed to provide lateral stability of the test subassembly. Additional lateral bracing of the test subassembly is not permitted, unless it replicates lateral bracing to be used in the prototype.

B-4.5 Essential Test Variables

The test specimen shall replicate as closely as is practical the pertinent design, detailing, construction features, and material properties of the prototype. The following variables shall be replicated in the test specimen.

B-4.5.1 Sources of Inelastic Rotation

Inelastic rotation shall be developed in the test specimen by inelastic action in the same members and connection elements as anticipated in the prototype (in other words, in the beam or link, in the column panel zone, in the column outside of the panel zone, or in connection elements) within the limits described below. The percentage of the total inelastic rotation in the test specimen that is developed in each member or connection element shall be within 25 percent of the anticipated percentage of the total inelastic rotation in the prototype that is developed in the corresponding member or connection element.

B-4.5.2 Size of Members

The size of the beam or link used in the test specimen shall be within the following limits:

1. The depth of the test beam or link shall be no less than 90 percent of the depth of the prototype beam or link.
2. The weight per foot of the test beam or link shall be no less than 75 percent of the weight per foot of the prototype beam or link.

The size of the column used in the test specimen shall properly represent the inelastic action in the column, as per the requirements in Section B-4.5.1. In addition, the depth of the test column shall be no less than 90 percent of the depth of the prototype column.

Extrapolation beyond the limitations stated in this Section shall be permitted subject to qualified peer review and approval by the authority having jurisdiction.

B-4.5.3 Connection Details

The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member sizes being tested.

B-4.5.4 Continuity Plates

The size and connection details of continuity plates used in the test specimen shall be proportioned to match the size and connection details of continuity plates used in the prototype connection as closely as possible.

Design Coefficients and Factors for basic Seismic Load Resisting Systems From (AISC)

B-4.5.5 Material Strength

The following additional requirements shall be satisfied for each member or connection element of the test specimen that supplies inelastic rotation by yielding:

1. The yield stress shall be determined by material tests on the actual materials used for the test specimen, as specified in Section B-4.8. The use of yield stress values that are reported on certified mill test reports are not permitted to be used for purposes of this Section.
2. The yield stress of the beam shall not be more than 15 percent below $R_y F_y$ for the grade of steel to be used for the corresponding elements of the prototype. Columns and connection elements with a tested yield stress shall not be more than 15 percent above or below $R_y F_y$ for the grade of steel to be used for the corresponding elements of the prototype. $R_y F_y$ shall be determined in accordance with Section 519.2.

B-4.5.6 Welds

Welds on the test specimen shall satisfy the following requirements:

1. Welding shall be performed in strict conformance with Welding Procedure Specifications (WPS) as required in AWS D1.1. The WPS essential variables shall meet the requirements in AWS D1.1 and shall be within the parameters established by the filler-metal manufacturer. The tensile strength of the welds used in the tested assembly and the Charpy V-Notch (CVN) toughness used in the tested assembly shall be determined by material tests as specified in Section B-4.8.3. The use of tensile strength and CVN toughness values that are

reported on the manufacturer's typical certificate of conformance is not permitted to be used for purposes of this section, unless the report includes results specific to Section B-7 requirements.

2. The specified minimum tensile strength of the filler metal used for the test specimen shall be the same as that to be used for the corresponding prototype welds. The tested tensile strength of the test specimen weld shall not be more than 125 MPa above the tensile strength classification of the filler metal specification specified for the prototype.
3. The specified minimum CVN toughness of the filler metal used for the test specimen shall not exceed the specified minimum CVN toughness of the filler metal to be used for the corresponding prototype welds. The tested CVN toughness of the test specimen weld shall not be more than 50 percent, nor 34 kJ, whichever is greater, above the minimum CVN toughness that will be specified for the prototype.
4. The welding positions used to make the welds on the test specimen shall be the same as those to be used for the prototype welds.
5. Details of weld backing, weld tabs, access holes, and similar items used for the test specimen welds shall be the same as those to be used for the corresponding prototype welds. Weld backing and weld tabs shall not be removed from the test specimen welds unless the corresponding weld backing and weld tabs are removed from the prototype welds.
6. Methods of inspection and nondestructive testing and standards of acceptance used for test specimen welds shall be the same as those to be used for the prototype welds.

B-4.5.7 Bolts

The bolted portions of the test specimen shall replicate the bolted portions of the prototype connection as closely as possible. Additionally, bolted portions of the test specimen shall satisfy the following requirements:

1. The bolt grade (for example, ASTM A325, A325M, ASTM A490, A490M, ASTM F1852) used in the test specimen shall be the same as that to be used for the prototype, except that ASTM A325 bolts may be substituted for ASTM F1852 bolts, and vice versa.
2. The type and orientation of bolt holes (standard, oversize, short slot, long slot, or other) used in the test specimen shall be the same as those to be used for the corresponding bolt holes in the prototype.
3. When inelastic rotation is to be developed either by yielding or by slip within a bolted portion of the connection, the method used to make the bolt holes (drilling, sub-punching and reaming, or other) in the test specimen shall be the same as that to be used in the corresponding bolt holes in the prototype.
4. Bolts in the test specimen shall have the same installation (pretensioned or other) and faying surface preparation (no specified slip resistance, Class A or B slip resistance, or other) as that to be used for the corresponding bolts in the prototype.

B-4.6 Loading History

B-4.6.1 General Requirements

The test specimen shall be subjected to cyclic loads according to the requirements prescribed in Section B-4.2 for beam-to-column moment connections in special and intermediate moment frames, and according to the requirements prescribed in Section B-4.3 for link-to-column connections in eccentrically braced frames.

Loading sequences other than those specified in Sections Section B-4.2 and Section B-4.3 may be used when they are demonstrated to be of equivalent or greater severity.

B-4.6.2 Loading Sequence for Beam-to-Column Moment Connections

Qualifying cyclic tests of beam-to-column moment connections in special and intermediate moment frames shall be conducted by controlling the interstory drift angle, θ , imposed on the test specimen, as specified below:

1. 6 cycles at $\theta = 0.00375$ rad
2. 6 cycles at $\theta = 0.005$ rad
3. 6 cycles at $\theta = 0.0075$ rad
4. 4 cycles at $\theta = 0.01$ rad
5. 2 cycles at $\theta = 0.015$ rad
6. 2 cycles at $\theta = 0.02$ rad
7. 2 cycles at $\theta = 0.03$ rad
8. 2 cycles at $\theta = 0.04$ rad

Continue loading at increments of $\theta = 0.01$ radian, with two cycles of loading at each step.

B-4.6.3 Loading Sequence for Link-to-Column Connections

Qualifying cyclic tests of link-to-column moment connections in eccentrically braced frames shall be conducted by controlling the total link rotation angle, γ_{total} , imposed on the test specimen, as follows:

1. 6 cycles at $\gamma_{total} = 0.00375$ rad
2. 6 cycles at $\gamma_{total} = 0.005$ rad
3. 6 cycles at $\gamma_{total} = 0.0075$ rad
4. 6 cycles at $\gamma_{total} = 0.01$ rad
5. 4 cycles at $\gamma_{total} = 0.015$ rad
6. 4 cycles at $\gamma_{total} = 0.02$ rad
7. 2 cycles at $\gamma_{total} = 0.03$ rad
8. 1 cycle at $\gamma_{total} = 0.04$ rad
9. 1 cycle at $\gamma_{total} = 0.05$ rad
10. 1 cycle at $\gamma_{total} = 0.07$ rad
11. 1 cycle at $\gamma_{total} = 0.09$ rad

Continue loading at increments of $\gamma_{total} = 0.02$ radian, with one cycle of loading at each step.

B-4.7 Instrumentation

Sufficient instrumentation shall be provided on the test specimen to permit measurement or calculation of the quantities listed in Section B-4.9.

B-4.8 Materials Testing Requirements

B-4.8.1 Tension Testing Requirements for Structural Steel

Tension testing shall be conducted on samples of steel taken from the material adjacent to each test specimen. Tension-test results from certified mill test reports shall be reported but are not permitted to be used in place of specimen testing for the purposes of this Section. Tension-test results shall be

based upon testing that is conducted in accordance with Section B-4.8.2.

Tension testing shall be conducted and reported for the following portions of the test specimen:

1. Flange(s) and web(s) of beams and columns at standard locations.
2. Any element of the connection that supplies inelastic rotation by yielding.

B-4.8.2 Methods of Tension Testing for Structural Steel

Tension testing shall be conducted in accordance with ASTM A6/A6M, ASTM A370, and ASTM E8, with the following exceptions:

1. The yield stress, F_y , that is reported from the test shall be based upon the yield strength definition in ASTM A370, using the offset method at 0.002 strain.
2. The loading rate for the tension test shall replicate, as closely as practical, the loading rate to be used for the test specimen.

B-4.8.3 Weld Metal Testing Requirements

The tensile strength of the welds used in the tested assembly and the CVN toughness used in the tested assembly shall be determined by material tests as specified in Section B-7. The use of tensile strength and CVN toughness values that are reported on the manufacturer's typical certificate of conformance is not permitted to be used for purposes of this section, unless that report includes results specific to Section B-7 requirements.

A single test plate may be used if the WPS for the test specimen welds is within plus/minus 0.8 kJ/mm of the WPS for the test plate.

Tensile specimens and CVN specimens shall be prepared in accordance with ANSI/AWS B4.0 Standard Methods for Mechanical Testing of Welds.

B-4.9 Test Reporting Requirements

For each test specimen, a written test report meeting the requirements of the authority having jurisdiction and the requirements of this Section shall be prepared. The report shall thoroughly document all key features and results of the test. The report shall include the following information:

1. A drawing or clear description of the test subassembly, including key dimensions, boundary

conditions at loading and reaction points, and location of lateral braces.

2. A drawing of the connection detail showing member sizes, grades of steel, the sizes of all connection elements, welding details including filler metal, the size and location of bolt holes, the size and grade of bolts, and all other pertinent details of the connection.
3. A listing of all other essential variables for the test specimen, as listed in Section B-4.
4. A listing or plot showing the applied load or displacement history of the test specimen.
5. A listing of all demand critical welds.
6. Definition of the region of the connection that comprises the protected zones.
7. A plot of the applied load versus the displacement of the test specimen. The displacement reported in this plot shall be measured at or near the point of load application. The locations on the test specimen where the loads and displacements were measured shall be clearly indicated.
8. A plot of beam moment versus interstory drift angle for beam-to-column moment connections; or a plot of link shear force versus link rotation angle for link-to-column connections. For beam-to-column connections, the beam moment and the interstory drift angle shall be computed with respect to the centerline of the column.
9. The interstory drift angle and the total inelastic rotation developed by the test specimen. The components of the test specimen contributing to the total inelastic rotation due to yielding or slip shall be identified. The portion of the total inelastic rotation contributed by each component of the test specimen shall be reported. The method used to compute inelastic rotations shall be clearly shown.
10. A chronological listing of significant test observations, including observations of yielding, slip, instability, and fracture of any portion of the test specimen as applicable.
11. The controlling failure mode for the test specimen. If the test is terminated prior to failure, the reason for terminating the test shall be clearly indicated.
12. The results of the material tests specified in Section B-4.8.
13. The Welding Procedure Specifications (WPS) and

welding inspection reports.

Additional drawings, data, and discussion of the test specimen or test results are permitted to be included in the report.

B-4.10 Acceptance Criteria

The test specimen must satisfy the strength and interstory drift angle or link rotation angle requirements of these Provisions for the special moment frame, intermediate moment frame, or eccentrically braced frame connection, as applicable. The test specimen must sustain the required interstory drift angle or link rotation angle for at least one complete loading cycle.

B-5. QUALIFYING CYCLIC TESTS OF BUCKLING-RESTRAINED BRACES

B-5.1 Scope

This appendix includes requirements for qualifying cyclic tests of individual buckling-restrained braces and buckling-restrained brace subassemblages, when required in these provisions. The purpose of the testing of individual braces is to provide evidence that a buckling-restrained brace satisfies the requirements for strength and inelastic deformation by these provisions; it also permits the determination of maximum brace forces for design of adjoining elements. The purpose of testing of the brace subassemblage is to provide evidence that the brace-design can satisfactorily accommodate the deformation and rotational demands associated with the design. Further, the subassemblage test is intended to demonstrate that the hysteretic behavior of the brace in the subassemblage is consistent with that of the individual brace elements tested uniaxially.

Alternative testing requirements are permitted when approved by the engineer-of-record and the authority having jurisdiction.

This appendix provides only minimum recommendations for simplified test conditions.

B-5.2 Symbols

The numbers in parentheses after the definition of a symbol refers to the Section number in which the symbol is first used.

- Δ_b = Deformation quantity used to control loading of the test specimen (total brace end rotation for the subassemblage test specimen; total brace axial deformation for the brace test specimen) (Section B-5.6).
- Δ_{bm} = Value of deformation quantity, Δ_b , corresponding to the design story drift (Section B-5.6).
- Δ_{by} = Value of deformation quantity, Δ_b , at first significant yield of test specimen (Section B-5.6).

B-5.3 Definitions

See Section for definitions.

B-5.4 Subassemblage Test Specimen

The subassemblage test specimen shall satisfy the following requirements:

1. The mechanism for accommodating inelastic rotation in the subassemblage test specimen brace shall be the same as that of the prototype. The rotational deformation demands on the subassemblage test specimen brace shall be equal to or greater than those of the prototype.
2. The axial yield strength of the steel core, P_{ysc} , of the brace in the subassemblage test specimen shall not be less than that of the prototype where both strengths are based on the core area, A_{sc} , multiplied by the yield strength as determined from a coupon test.
3. The cross-sectional shape and orientation of the steel core projection of the subassemblage test specimen brace shall be the same as that of the brace in the prototype.
4. The same documented design methodology shall be used for design of the subassemblage as used for the prototype, to allow comparison of the rotational deformation demands on the subassemblage brace to the prototype. In stability calculations, beams, columns, and gussets connecting the core shall be considered parts of this system.
5. The calculated margins of safety for the prototype connection design, steel core projection stability, overall buckling and other relevant subassemblage test specimen brace construction details, excluding the gusset plate, for the prototype, shall equal or exceed those of the subassemblage test specimen construction.
6. Lateral bracing of the subassemblage test specimen shall replicate the lateral bracing in the prototype.
7. The brace test specimen and the prototype shall be manufactured in accordance with the same quality control and assurance processes and procedures.

Extrapolation beyond the limitations stated in this section shall be permitted subject to qualified peer review and approval by the authority having jurisdiction.

B-5.5 Brace Test Specimen

The brace test specimen shall replicate as closely as is practical the pertinent design, detailing, construction features, and material properties of the prototype.

B-5.5.1 Design of Brace Test Specimen

The same documented design methodology shall be used for the brace test specimen and the prototype. The design calculations shall demonstrate, at a minimum, the following requirements:

1. The calculated margin of safety for stability against overall buckling for the prototype shall equal or exceed that of the brace test specimen.
2. The calculated margins of safety for the brace test specimen and the prototype shall account for differences in material properties, including yield and ultimate stress, ultimate elongation, and toughness.

B-5.5.2 Manufacture of Brace Test Specimen

The brace test specimen and the prototype shall be manufactured in accordance with the same quality control and assurance processes and procedures.

B-5.5.3 Similarity of Brace Test Specimen and Prototype

The brace test specimen shall meet the following requirements:

1. The cross-sectional shape and orientation of the steel core shall be the same as that of the prototype.
2. The axial yield strength of the steel core, P_{ySC} , of the brace test specimen shall not vary by more than 50 percent from that of the prototype where both strengths are based on the core area, A_{SC} , multiplied by the yield strength as determined from a coupon test.
3. The material for, and method of, separation between the steel core and the buckling restraining mechanism in the brace test specimen shall be the same as that in the prototype.

Extrapolation beyond the limitations stated in this section shall be permitted subject to qualified peer review and approval by the authority having jurisdiction.

B-5.5.4 Connection Details

The connection details used in the brace test specimen shall represent the prototype connection details as closely as practical.

B-5.5.5 Materials

1. Steel core: The following requirements shall be satisfied for the steel core of the brace test specimen:
 - a. The specified minimum yield stress of the brace test specimen steel core shall be the same as that of the prototype.
 - b. The measured yield stress of the material of the steel core in the brace test specimen shall be at least 90 percent of that of the prototype as determined from coupon tests.
 - c. The specified minimum ultimate stress and strain of the brace test specimen steel core shall not exceed those of the prototype.

2. Buckling-restraining mechanism

Materials used in the buckling-restraining mechanism of the brace test specimen shall be the same as those used in the prototype.

B-5.5.6 Connections

The welded, bolted, and pinned joints on the test specimen shall replicate those on the prototype as close as practical.

B-5.6 Loading History**B-5.6.1 General Requirements**

The test specimen shall be subjected to cyclic loads according to the requirements prescribed in Sections B-5.6.2 and B-5.6.3. Additional increments of loading beyond those described in Section B-5.6.3 are permitted. Each cycle shall include a full tension and full compression excursion to the prescribed deformation.

B-5.6.2 Test Control

The test shall be conducted by controlling the level of axial or rotational deformation, Δ_b , imposed on the test specimen. As an alternate, the maximum rotational deformation may be applied and maintained as the protocol is followed for axial deformation.

B-5.6.3 Loading Sequence

Loads shall be applied to the test specimen to produce the following deformations, where the deformation is the steel core axial deformation for the test specimen and the rotational deformation demand for the subassembly test specimen brace:

1. 2 cycles of loading at the deformation corresponding to $\Delta_b = \Delta b_y$
2. 2 cycles of loading at the deformation corresponding to $\Delta_b = 0.50\Delta b_m$
3. 2 cycles of loading at the deformation corresponding to $\Delta_b = 1\Delta b_m$
4. 2 cycles of loading at the deformation corresponding to $\Delta_b = 1.5\Delta b_m$
5. 2 cycles of loading at the deformation corresponding to $\Delta_b = 2.0\Delta b_m$
6. Additional complete cycles of loading at the deformation corresponding to $\Delta_b = 1.5\Delta b_m$ as required for the brace test specimen to achieve a cumulative inelastic axial deformation of at least 200 times the yield deformation (not required for the subassembly test specimen).

The design story drift shall not be taken as less than 0.01 times the story height for the purposes of calculating Δb_m . Other loading sequences are permitted to be used to qualify the test specimen when they are demonstrated to be of equal or greater severity in terms of maximum and cumulative inelastic deformation.

B-5.7 Instrumentation

Sufficient instrumentation shall be provided on the test specimen to permit measurement or calculation of the quantities listed in Section B-5.9.

B-5.8 Materials Testing Requirements

B-5.8.1 Tension Testing Requirements

Tension testing shall be conducted on samples of steel taken from the same material as that used to manufacture the steel core. Tension test results from certified mill test reports shall be reported but are not permitted to be used in place of specimen testing for the purposes of this Section. Tension-test results shall be based upon testing that is conducted in accordance with Section B-5.8.2.

B-5.8.2 Methods of Tension Testing

Tension testing shall be conducted in accordance with ASTM A6, ASTM A370, and ASTM E8, with the following exceptions:

1. The yield stress that is reported from the test shall be based upon the yield strength definition in ASTM A370, using the offset method of 0.002 strain.
2. The loading rate for the tension test shall replicate, as closely as is practical, the loading rate used for the test specimen.
3. The coupon shall be machined so that its longitudinal axis is parallel to the longitudinal axis of the steel core.

B-5.9 Test Reporting Requirements

For each test specimen, a written test report meeting the requirements of this Section shall be prepared. The report shall thoroughly document all key features and results of the test. The report shall include the following information:

1. A drawing or clear description of the test specimen, including key dimensions, boundary conditions at loading and reaction points, and location of lateral bracing, if any.
2. A drawing of the connection details showing member sizes, grades of steel, the sizes of all connection elements, welding details including filler metal, the size and location of bolt or pin holes, the size and grade of connectors, and all other pertinent details of the connections.
3. A listing of all other essential variables as listed in Section B-5.4 or B-5.5, as appropriate.
4. A listing or plot showing the applied load or displacement history.
5. A plot of the applied load versus the deformation, Δb . The method used to determine the deformations shall be clearly shown. The locations on the test specimen where the loads and deformations were measured shall be clearly identified.
6. A chronological listing of significant test observations, including observations of yielding, slip, instability, transverse displacement along the test specimen and fracture of any portion of the test specimen and connections, as applicable.
7. The results of the material tests specified in Section B-5.8.

8. The manufacturing quality control and quality assurance plans used for the fabrication of the test specimen. These shall be included with the welding procedure specifications and welding inspection reports.

Additional drawings, data, and discussion of the test specimen or test results are permitted to be included in the report.

B-5.10 Acceptance Criteria

At least one subassembly test that satisfies the requirements of Section B-5.4 shall be performed. At least one brace test that satisfies the requirements of Section B-4.5 shall be performed. Within the required protocol range all tests shall satisfy the following requirements:

1. The plot showing the applied load vs. displacement history shall exhibit stable, repeatable behavior with positive incremental stiffness.
2. There shall be no fracture, brace instability or brace end connection failure.
3. For brace tests, each cycle to a deformation greater than Δb_y the maximum tension and compression forces shall not be less than the nominal strength of the core.
4. For brace tests, each cycle to a deformation greater than Δb_y the ratio of the maximum compression force to the maximum tension force shall not exceed 1.3.

Other acceptance criteria may be adopted for the brace test specimen or subassembly test specimen subject to qualified peer review and approval by the authority having jurisdiction.

B-6. WELDING PROVISIONS

B-6.1 Scope

This appendix provides additional details regarding welding and welding inspection, and is included on an interim basis pending adoption of such criteria by AWS or other accredited organization.

B-6.2 Structural Design Drawings and Specifications, Shop Drawings, and Erection Drawings

B-6.2.1 Structural Design Drawings and Specifications

Structural design drawings and specifications shall include, as a minimum, the following information:

1. Locations where backup bars are required to be removed.
2. Locations where supplemental fillet welds are required when backing is permitted to remain.
3. Locations where fillet welds are used to reinforce groove welds or to improve connection geometry.
4. Locations where weld tabs are required to be removed.
5. Splice locations where tapered transitions are required.

User Note: Butt splices subject to tension greater than 33 percent of the expected yield strength under any load combination should have tapered transitions. The stress concentration at a nontapered transition, based upon a 90° corner, could cause local yielding when the tensile stress exceeds 33 percent of yield. Lower levels of stress would be acceptable with the stress concentration from a nontapered transition.

6. The shape of weld access holes, if a special shape is required.
7. Joints or groups of joints in which a specific assembly order, welding sequence, welding technique or other special precautions are required.

B-6.2.2 Shop Drawings

Shop drawings shall include, as a minimum, the following information:

1. Access hole dimensions, surface profile and finish requirements.
2. Locations where backing bars are to be removed.
3. Locations where weld tabs are to be removed.
4. NDT to be performed by the fabricator, if any.

B-6.2.3 Erection Drawings

Erection drawings shall include, as a minimum, the following information:

1. Locations where backing bars to be removed.
2. Locations where supplemental fillets are required when backing is permitted to remain.
3. Locations where weld tabs are to be removed.
4. Those joints or groups of joints in which a specific assembly order, welding sequence, welding technique or other special precautions are required.

B-6.3 Personnel**B-6.3.1 QC Welding Inspectors**

QC welding inspection personnel shall be associate welding inspectors (AWI) or higher, as defined in AWS B5.1 Standard for the Qualification of Welding Inspectors, or otherwise qualified under the provisions of AWS D1.1 Section 6.1.4 and to the satisfaction of the contractor's QC plan by the fabricator/erector.

B-6.3.2 QA Welding Inspectors

QA welding inspectors shall be welding inspectors (WI), or senior welding inspectors (SWI), as defined in AWS B5.1, except AWIs may be used under the direct supervision of WIs, on site and available when weld inspection is being conducted.

B-6.3.3 Nondestructive Testing Technicians

NDT technicians shall be qualified as follows:

1. In accordance with their employer's written practice which shall meet or exceed the criteria of the American Society for Nondestructive Testing, Inc. SNT TC-1A Recommended Practice for the Training and Testing of Nondestructive Personnel, or of ANSI/ASNT CP-189, Standard for the Qualification and Certification of Nondestructive Testing Personnel.
2. Ultrasonic Testing for QA may be performed only by UT technicians certified as ASNT Level III through examination by the ASNT, or certified as Level II by their employer for flaw detection. If the engineer-of-record approves the use of flaw sizing techniques, UT technicians shall also be qualified and certified by their employer for flaw sizing.
3. Magnetic Particle Testing (MT) and Dye Penetrant Testing (PT) for QA may be performed only by technicians certified as Level II by their employer, or certified as ASNT Level III through examination by the ASNT and certified by their employer.

B-6.4 Nondestructive Testing Procedures**B-6.4.1 Ultrasonic Testing**

Ultrasonic Testing shall be performed according to the procedures prescribed in AWS D1.1 Section 6, Part F following a written procedure containing the elements prescribed in paragraph K3 of Annex K. Section 6, Part F procedures shall be qualified using weld mock-ups having 1.5 mm-diameter side drilled holes similar to Annex K, Figure K-3.

B-6.4.2 Magnetic Particle

Magnetic particle testing shall be performed according to procedures prescribed in AWS D1.1, following a written procedure utilizing the Yoke Method that conforms to ASTM E709.

B-6.5 Additional Welding Provisions

B-6.5.1 Intermixed Filler Metals

When FCAW-S filler metals are used in combination with filler metals of other processes, including FCAW-G, a test specimen shall be prepared and mechanical testing shall be conducted to verify that the notch toughness of the combined materials in the intermixed region of the weld meets the notch toughness requirements of Section 520.3.1 and, if required, the notch toughness requirements for demand critical welds of Section 520.3.2.

B-6.5.2 Filler Metal Diffusible Hydrogen

Welding electrodes and electrode-flux combinations shall meet the requirements for H16 (16 mL maximum diffusible hydrogen per 100 grams deposited weld metal) as tested in accordance with AWS A4.3 Standard Methods for Determination of the Diffusible Hydrogen Content of Martensitic, Bainitic, and Ferritic Steel Weld Metal Produced by Arc Welding. (Exception: GMAW solid electrodes.) The manufacturer's typical certificate of conformance shall be considered adequate proof that the supplied electrode or electrode-flux combination meets this requirement. No testing of filler metal samples or of production welds shall be required.

B-6.5.3 Gas-Shielded Welding Processes

GMAW and FCAW-G shall not be performed in winds exceeding 5 km/hr. Windscreens or other shelters may be used to shield the welding operation from excessive wind.

B-6.5.4 Maximum Interpass Temperatures

Maximum interpass temperatures shall not exceed 290 °C, measured at a distance not exceeding 75 mm from the start of the weld pass. The maximum interpass temperature may be increased by qualification testing that includes weld metal and base metal CVN testing using AWS D1.1 Annex III. The steel used for the qualification testing shall be of the same type and grade as will be used in production.

The maximum heat input to be used in production shall be used in the qualification testing. The qualified maximum interpass temperature shall be the lowest interpass temperature used for any pass during qualification testing. Both weld metal and HAZ shall be tested. The weld metal shall meet all the mechanical properties required by Section 520.3.1, or those for demand critical welds of Section 520.3.2, as applicable. The heat affected zone CVN toughness shall meet a minimum requirement of 27 J at 21 °C with specimens taken at both 1 and 5 mm from the fusion line.

B-6.5.5 Weld Tabs

Where practicable, weld tabs shall extend beyond the edge of the joint a minimum of one inch or the thickness of the part, whichever is greater. Extensions need not exceed 50 mm.

Where used, weld tabs shall be removed to within 3 mm of the base metal surface, except at continuity plates where removal to within 6 mm of the plate edge is acceptable, and the end of the weld finished. Removal shall be by air carbon arc cutting (CAC-A), grinding, chipping, or thermal cutting. The process shall be controlled to minimize errant gouging. The edges where weld tabs have been removed shall be finished to a surface roughness of **13 μ m** or better. Grinding to a flush condition is not required. The contour of the weld end shall provide a smooth transition, free of notches and sharp corners. At T-joints, a minimum radius in the corner need not be provided. The weld end shall be free of gouges and notches. Weld defects not greater than 2 mm deep shall be faired to a slope not greater than 1:5. Other weld defects shall be excavated and repaired by welding in accordance with an applicable WPS.

B-6.5.6 Bottom Flange Welding Sequence

When using weld access holes to facilitate CJP groove welds of beam bottom flanges to column flanges or continuity plates, the groove weld shall be sequenced as follows:

1. As far as is practicable, starts and stops shall not be placed directly under the beam web.
2. Each layer shall be completed across the full width of the flange before beginning the next layer.
3. For each layer, the weld starts and stops shall be on the opposite side of the beam web, as compared to the previous layer.

B-6.6 Additional Welding Provisions for Demand Critical Welds Only

B-6.6.1 Welding Processes

SMAW, GMAW (except short circuit transfer), FCAW and SAW may be used to fabricate and erect members governed by this specification. Other processes may be used, provided that one or more of the following criteria is met:

1. The process is part of the prequalified connection details, as listed in Section B-1,
2. The process was used to perform a connection qualification test in accordance with Section B-4, or

3. The process is approved by the engineer-of-record.

B-6.6.2 Filler Metal Packaging

Electrodes shall be provided in packaging that limits the ability of the electrode to absorb moisture. Electrode from packaging that has been punctured or torn shall be dried in accordance with the manufacturer's recommendations, or shall not be used for demand critical welds.

Modification or lubrication of the electrode after manufacture is prohibited, except that drying is permitted as recommended by the manufacturer.

B-6.6.3 Exposure Limitations on FCAW Electrodes

After removal from protective packaging, the permissible atmospheric exposure time of FCAW electrodes shall be limited as follows:

1. Exposure shall not exceed the electrode manufacturer's guidelines.
2. In the absence of manufacturer's recommendations, the total accumulated exposure time for FCAW electrodes shall not exceed 72 hours. When the electrodes are not in use, they may be stored in protective packaging or a cabinet. Storage time shall not be included in the accumulated exposure time. Electrodes that have been exposed to the atmosphere for periods exceeding the above time limits shall be dried in accordance with the electrode manufacturer's recommendations, or shall not be used for demand critical welds. The electrode manufacturer's recommendations shall include time, temperature, and number of drying cycles permitted.

B-6.6.4 Tack Welds

Tack welds attaching backing bars and weld tabs shall be placed where they will be incorporated into a final weld.

B-7 WELD METAL/WELDING PROCEDURE SPECIFICATION NOTCH TOUGHNESS VERIFICATION TEST

This appendix provides a procedure for qualifying the weld metal toughness and is included on an interim basis pending adoption of such a procedure by the American Welding Society (AWS) or other accredited organization.

B-7.1 Scope

This appendix provides a standard method for qualification testing of weld filler metals required to have specified notch toughness for service in joints designated as demand critical.

Testing of weld metal to be used in production shall be performed by filler metal manufacturer's production lot, as defined in AWS A5.01, Filler Metal Procurement Guidelines, as follows:

1. Class C3 for SMAW electrodes,
2. Class S2 for GMAW-S and SAW electrodes,
3. Class T4 for FCAW and GMAW-C, or
4. Class F2 for SAW fluxes.

Filler metals produced by manufacturers audited and approved by one or more of the following agencies shall be exempt from these production lot testing requirements, provided a minimum of 3 production lots of material, as defined above, are tested in accordance with the provisions of this appendix:

1. American Bureau of Shipping (ABS),
2. Lloyds Register of Shipping,
3. American Society of Mechanical Engineers (ASME),
4. ISO 9000,
5. US Department of Defense, or
6. A quality assurance program acceptable to the engineer-of-record.

Under this exemption from production lot testing, the filler metal manufacturer shall repeat the testing prescribed in this appendix at least every three years on a random production lot.

B-7.2 Test Conditions

Tests shall be conducted at the range of heat inputs for which the weld filler metal will be qualified under the Welding Procedure Specification (WPS). It is recommended that tests be conducted at the low heat input level and high heat input level indicated in Table B-7.2-1.

Table B-7.2-1 WPS Toughness Verification Test
Welding and Preheat Conditions

Cooling Rate	Heat Input	Preheat °F (°C)	Interpass °F (°C)
Low heat input test	30 kJ/in. (1.2kJ/mm)	70±25 (21±14)	200±50 (21±14)
High heat input test	80 kJ/in. (1.2kJ/mm)	300±25 (149±14)	500±50 (260±28)

Alternatively, the filler metal manufacturer or contractor may elect to test a wider or narrower range of heat inputs and interpass temperatures. The range of heat inputs and interpass temperatures tested shall be clearly stated on the test reports and user data sheets. Regardless of the method of selecting test heat input, the WPS, as used by the contractor, shall fall within the range of heat inputs and interpass temperatures tested.

B-7.3 Test Specimens

Two test plates, one for each heat input, shall be welded following Table B-7.2-1. Five CVN specimens and one tensile specimen shall be prepared per plate. Each plate shall be steel, of any AISC-listed structural grade. The test plate shall be 19 mm thick with a 13 mm root opening and 45° included groove angle. The test plate and specimens shall be as shown in Figure 2A in AWS A5.20, or as in Figure 5 in AWS A5.29. Except for the root pass, a minimum of two passes per layer shall be used to fill the width.

All test specimens shall be taken from near the centerline of the weld at the mid-thickness location, in order to minimize dilution effects. CVN and tensile specimens shall be prepared in accordance with AWS B4.0, Standard Methods for Mechanical Testing of Welds. The test assembly shall be restrained during welding, or preset at approximately 5° to prevent warpage in excess of 5°. A welded test assembly that has warped more than 5° shall be discarded. Welded test assemblies shall not be straightened.

The test assembly shall be tack welded and heated to the specified preheat temperature, measured by temperature indicating crayons or surface temperature thermometers one inch from the center of the groove at the location shown in the figures cited above. Welding shall continue until the assembly has reached the interpass temperature prescribed in Table B-7.2-1. The interpass temperature shall be maintained for the remainder of the weld. Should it be necessary to interrupt welding, the assembly shall be allowed to cool in air. The assembly shall then be heated to the prescribed interpass temperature before welding is resumed.

No thermal treatment of weldment or test specimens is permitted, except that machined tensile test specimens may be aged at 200 °F (93 °C) to 220 °F (104 °C) for up to 48 hours, then cooled to room temperature before testing.

B-7.4 Acceptance Criteria

The lowest and highest Charpy V-Notch (CVN) toughness values obtained from the five specimens from a single test plate shall be disregarded. Two of the remaining three values shall equal, or exceed, the specified toughness of 54 J energy level at the testing temperature. One of the three may be lower, but not lower than 41 J, and the average of the three shall not be less than the required 54 J energy level. All test samples shall meet the notch toughness requirements for the electrodes as provided in Section 520.3.2.

For filler metals classified as E70, materials shall provide a minimum yield stress of 58 ksi, a minimum tensile strength of 70 ksi, and a minimum elongation of 22 percent. For filler metals classified as E80, materials shall provide a minimum yield stress of 68 ksi, a minimum tensile strength of 80 ksi, and a minimum elongation of 19 percent.

PART 2B COMPOSITE STRUCTURAL STEEL AND REINFORCED CONCRETE BUILDINGS

SECTION 532 SCOPE

These Provisions shall govern the design, fabrication, and erection of composite structural steel and reinforced concrete members and connections in the Seismic Load Resisting Systems (SLRS) in buildings and other structures, where other structures are defined as those designed, fabricated, and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting systems. These provisions shall apply when the seismic response modification coefficient, **R**, (as specified in the NSCP code) is taken greater than 3.

When the seismic response modification coefficient, **R**, is taken as 3 or less, the structure is not required to satisfy these provisions unless required by the NSCP code.

The requirements of Part 2B modify and supplement the requirements of Part 2A and form these Provisions. They shall be applied in conjunction with the AISC Specification for Structural Steel Buildings, ANSI/AISC 360, hereinafter referred to as the Specification. The applicable requirements of the Building Code Requirements for Structural Concrete and Commentary, ACI 318, as modified in these Provisions shall be used for the design of reinforced concrete components in composite SLRS.

For seismic load resisting systems incorporating reinforced concrete components designed according to ACI 318, the requirements for load and resistance factor design as specified in Section 502.3 of the Specification shall be used.

When the design is based upon elastic analysis, the stiffness properties of the component members of composite systems shall reflect their condition at the onset of significant yielding of the structure.

Wherever these Provisions refer to the NSCP code and there is no local building code, the loads, load combinations, system limitations and general design requirements shall be those in SEI/ASCE 7.

Part 2B includes a Glossary which is specifically applicable to this Part. The Part 2A Glossary is also applicable to Part 2B.

SECTION 533 REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

The documents referenced in these provisions shall include those listed in Part 2A Section 515 with the following additions:

American Society of Civil Engineers Standard for the Structural Design of Composite Slabs, ASCE 3-91

American Welding Society Structural Welding Code-Reinforcing Steel, AWS D1.4-98

SECTION 534 GENERAL SEISMIC DESIGN REQUIREMENTS

The required strength and other provisions for Seismic Design Categories (SDCs) and seismic use groups and the limitations on height and irregularity shall be as specified in the NSCP code.

The design story drift and story drift limits shall be determined as required in the NSCP code.

SECTION 535 LOADS, LOAD COMBINATIONS, AND NOMINAL STRENGTHS

535.1 Loads and Load Combinations

Where amplified seismic loads are required by these Provisions, the horizontal portion of the earthquake load E (as defined in the NSCP code) shall be multiplied by the overstrength factor Ω_o prescribed by the NSCP code.

For the Seismic Load Resisting System (SLRS) incorporating reinforced concrete components designed according to ACI 318, the requirements of Section 502.3 of the Specification shall be used.

User Note: When not defined in the NSCP code, Ω_o should be taken from SEI/ASCE 7.

535.2 Nominal Strength

The nominal strength of systems, members, and connections shall be determined in accordance with the requirements of the Specification, except as modified throughout these Provisions.

SECTION 536 MATERIALS

536.1 Structural Steel

Structural steel members and connections used in composite Seismic Load Resisting Systems (SLRS) shall meet the requirements of Specification Section 501.3. Structural steel used in the composite SLRS described in Sections 539, 540, 543, 545, 547 and 548 shall also meet the requirements in Part 2A Sections 519 and 520.

536.2 Concrete and Steel Reinforcement

Concrete and steel reinforcement used in composite components in composite SLRS shall meet the requirements of ACI 318, Sections 21.2.4 through 21.2.8.

Exception:

Concrete and steel reinforcement used in the composite ordinary seismic systems described in Sections 542, 544, and 546 shall meet the requirements of Section 509 and ACI 318, excluding Chapter 21.

SECTION 537 COMPOSITE MEMBERS

537.1 Scope

The design of composite members in the SLRS described in Sections 539 through 548 shall meet the requirements of this Section and the material requirements of Section 536.

537.2 Composite Floor and Roof Slabs

The design of composite floor and roof slabs shall meet the requirements of ASCE 3. Composite slab diaphragms shall meet the requirements in this Section.

537.2.1 Load Transfer

Details shall be designed so as to transfer loads between the diaphragm and boundary members, collector elements, and elements of the horizontal framing system.

537.2.2 Nominal Shear Strength

The nominal shear strength of composite diaphragms and concrete-filled steel deck diaphragms shall be taken as the nominal shear strength of the reinforced concrete above the top of the steel deck ribs in accordance with ACI 318 excluding Chapter 22. Alternatively, the composite diaphragm nominal shear strength shall be determined by in-plane shear tests of concrete-filled diaphragms.

537.3 Composite Beams

Composite beams shall meet the requirements of Section 509. Composite beams that are part of Composite-Special Moment Frames (C-SMF) shall also meet the requirements of Section 540.3.

537.4 Encased Composite Columns

This section is applicable to columns that:

1. Consist of reinforced-concrete encased shapes with a structural steel area that comprises at least 1 percent of the total composite column cross section; and
2. Meet the additional limitations of Specification Section 509.2.1. Such columns shall meet the requirements of Specification Section 509, except as modified in this Section. Additional requirements, as specified for intermediate and special seismic systems in Sections 537.4.2 and 537.4.3 shall apply as required in the descriptions of the composite seismic systems in Sections 539 through 548.

Columns that consist of reinforced-concrete-encased shapes shall meet the requirements for reinforced concrete columns of ACI 318 except as modified for:

1. The structural steel section shear connectors in Section 537.4.2.
2. The contribution of the reinforced-concrete-encased shape to the strength of the column as provided in ACI 318.
3. The seismic requirements for reinforced concrete columns as specified in the description of the composite seismic systems in Sections 539 through 548.

537.4.1 Ordinary Seismic System Requirements

The following requirements for encased composite columns are applicable to all composite systems, including ordinary seismic systems:

1. The available shear strength of the column shall be determined in accordance with Specification Section 509.2.1.1d. The nominal shear strength of the tie reinforcement shall be determined in accordance with ACI 318 Sections 11.5.6.2 through 11.5.6.9. In ACI 318 Sections 11.5.6.5 and 11.5.6.9, the dimension b_w shall equal the width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear.
2. Composite columns designed to share the applied loads between the structural steel section and the reinforced concrete encasement shall have shear connectors that meet the requirements of Specification Section 509.2.1.
3. The maximum spacing of transverse ties shall meet the requirements of Specification Section 509.2.1.

Transverse ties shall be located vertically within one-half of the tie spacing above the top of the footing or lowest beam or slab in any story and shall be spaced as provided herein within one-half of the tie spacing below the lowest beam or slab framing into the column.

Transverse bars shall have a diameter that is not less than one-fiftieth of the greatest side dimension of the composite member, except that ties shall not be smaller than Diameter 10mm bars and need not be larger than Diameter 16 mm bars. Alternatively, welded wire fabric of equivalent area is permitted as transverse reinforcement except when prohibited for intermediate and special seismic systems.

4. Load-carrying reinforcement shall meet the detailing and splice requirements of ACI 318 Sections 7.8.1 and 12.17. Load-carrying reinforcement shall be provided at every corner of a rectangular cross-section. The maximum spacing of other load carrying or restraining longitudinal reinforcement shall be one-half of the least side dimensions of the composite member.
5. Splices and end bearing details for encased composite columns in ordinary seismic systems shall meet the requirements of the Specification and ACI 318 Section 7.8.2. The design shall comply with ACI 318 Sections 21.2.6, 21.2.7 and 21.10. The design shall consider any adverse behavioral effects due to abrupt changes in either the member stiffness or the nominal tensile strength. Such locations shall include transitions to reinforced concrete sections without embedded structural steel members, transitions to bare structural steel sections, and column bases.

537.4.2 Intermediate Seismic System Requirements

Encased composite columns in intermediate seismic systems shall meet the following requirements in addition to those of Section 537.4.1:

1. The maximum spacing of transverse bars at the top and bottom shall be the least of the following:
 - a. one-half the least dimension of the section
 - b. 8 longitudinal bar diameters
 - c. 24 tie bar diameters
 - d. 300 mm

These spacings shall be maintained over a vertical distance equal to the greatest of the following lengths, measured from each joint face and on both sides of any section where flexural yielding is expected to occur:

- a. one-sixth the vertical clear height of the column
 - b. the maximum cross-sectional dimension
 - c. 450 mm
2. Tie spacing over the remaining column length shall not exceed twice the spacing defined above.
 3. Welded wire fabric is not permitted as transverse reinforcement in intermediate seismic systems.

537.4.3 Special Seismic System Requirements

Encased composite columns in special seismic systems shall meet the following requirements in addition to those of Sections 537.4.1 and 537.4.2:

1. The required axial strength for encased composite columns and splice details shall meet the requirements in Part 2A Section 521.3.
2. Longitudinal load-carrying reinforcement shall meet the requirements of ACI 318 Section 21.4.3.
3. Transverse reinforcement shall be hoop reinforcement as defined in ACI 318 Chapter 21 and shall meet the following requirements:
 - a. The minimum area of tie reinforcement A_{sh} shall meet the following:

$$A_{sh} = 0.09h_{cc}s \left(1 - \frac{F_y A_s}{P_n} \right) \left(\frac{f'_c}{F_{yh}} \right) \quad (537-1)$$

where

h_{cc} = cross-sectional dimension of the confined core measured center-to-center of the tie reinforcement, mm

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, mm

F_y = specified minimum yield stress of the structural steel core, MPa

A_s = cross-sectional area of the structural core, mm²

P_n = nominal compressive strength of the composite column calculated in accordance with the Specification, N

f'_c = specified compressive strength of concrete, MPa

F_{yh} = specified minimum yield stress of the ties, MPa

Eq. 537-1 need not be satisfied if the nominal strength of the reinforced-concrete-encased structural steel section alone is greater than the load effect from a load combination of **1.0 D + 0.5L**.

- b. The maximum spacing of transverse reinforcement along the length of the column shall be the lesser of six longitudinal load-carrying bar diameters or 150 mm.

- c. When specified in Sections 537.4.3(4), 537.4.3 (5) or 537.4.3 (6), the maximum spacing of transverse reinforcement shall be the lesser of one-fourth the least member dimension or 100 mm. For this reinforcement, cross ties, legs of overlapping hoops, and other confining reinforcement shall be spaced not more than 350 mm on center in the transverse direction.
4. Encased composite columns in braced frames with nominal compressive loads that are larger than 0.2 times P_n shall have transverse reinforcement as specified in Section 537.4.3 (3)(iii) over the total element length. This requirement need not be satisfied if the nominal strength of the reinforced-concrete-encased steel section alone is greater than the load effect from a load combination of **1.0 D + 0.5L**.
5. Composite columns supporting reactions from discontinued stiff members, such as walls or braced frames, shall have transverse reinforcement as specified in Section 537.4.3 (3)(iii) over the full length beneath the level at which the discontinuity occurs if the nominal compressive load exceeds 0.1 times P_n . Transverse reinforcement shall extend into the discontinued member for at least the length required to develop full yielding in the reinforced-concrete-encased shape and longitudinal reinforcement. This requirement need not be satisfied if the nominal strength of the reinforced-concrete encased structural steel section alone is greater than the load effect from a load combination of **1.0 D + 0.5L**.
6. Encased composite columns used in a C-SMF shall meet the following requirements:
 - a. Transverse reinforcement shall meet the requirements in Section 537.4.3 (3)(c) at the top and bottom of the column over the region specified in Section 537.4.2.
 - b. The strong - column / weak - beam design requirements in Section 540.5 shall be satisfied. Column bases shall be detailed to sustain inelastic flexural hinging.
 - c. The required shear strength of the column shall meet the requirements of ACI 318 Section 21.4.5.1.

7. When the column terminates on a footing or mat foundation, the transverse reinforcement as specified in this section shall extend into the footing or mat at least 300 mm. When the column terminates on a wall, the transverse reinforcement shall extend into the wall for at least the length required to develop full yielding in the reinforced-concrete-encased shape and longitudinal reinforcement.
8. Welded wire fabric is not permitted as transverse reinforcement for special seismic systems.

537.5 Filled Composite Columns

This Section is applicable to columns that meet the limitations of Specification Section 509.2.2. Such columns shall be designed to meet the requirements of Specification Section 509, except as modified in this Section.

1. The nominal shear strength of the composite column shall be the nominal shear strength of the structural steel section alone, based on its effective shear area. The concrete shear capacity may be used in conjunction with the shear strength from the steel shape provided the design includes an appropriate load transferring mechanism.
2. In addition to the requirements of Section 537.5(1), in the special seismic systems described in Sections 540, 543 and 545, the design loads and column splices for filled composite columns shall also meet the requirements of Part 2a Section 521.
3. Filled composite columns used in C-SMF shall meet the following requirements in addition to those of Sections 6.5(1) and 6.5(2):
 - a. The minimum required shear strength of the column shall meet the requirements in ACI 318 Section 21.4.5.1.
 - b. The strong-column/weak-beam design requirements in Section 521.5 shall be met. Column bases shall be designed to sustain inelastic flexural hinging.
 - c. The minimum wall thickness of concrete-filled rectangular HSS shall be:

$$t_{min} = \sqrt{F_y/2E} \quad (537-2)$$

for the flat width b of each face, where b is as defined in Specification Table 502.4.1.

SECTION 538 COMPOSITE CONNECTIONS

538.1 Scope

This Section is applicable to connections in buildings that utilize composite or dual steel and concrete systems wherein seismic load is transferred between structural steel and reinforced concrete components.

Composite connections shall be demonstrated to have strength, ductility and toughness comparable to that exhibited by similar structural steel or reinforced concrete connections that meet the requirements of Part 2A and ACI 318, respectively. Methods for calculating the connection strength shall meet the requirements in this Section.

538.2 General Requirements

Connections shall have adequate deformation capacity to resist the required strength at the design story drift. Additionally, connections that are required for the lateral stability of the building under seismic loads shall meet the requirements in Sections 539 through 548 based upon the specific system in which the connection is used. When the available strength of the connected members is based upon nominal material strengths and nominal dimensions, the determination of the available strength of the connection shall account for any effects that result from the increase in the actual nominal strength of the connected member.

538.3 Nominal Strength of Connections

The nominal strength of connections in composite structural systems shall be determined on the basis of rational models that satisfy both equilibrium of internal forces and the strength limitation of component materials and elements based upon potential limit states. Unless the connection strength is determined by analysis and testing, the models used for analysis of connections shall meet the requirements of Sections 538.3(1) through 538.3(5).

1. When required, force shall be transferred between structural steel and reinforced concrete through
 - (a) direct bearing of headed shear studs or suitable alternative devices;
 - (b) by other mechanical means;
 - (c) by shear friction with the necessary clamping force provided by reinforcement normal to the plane of shear transfer; or
 - (d) by a combination of these means.
 Any potential bond strength between structural steel and reinforced concrete shall be ignored for the purpose of the connection force transfer mechanism. The contribution of different mechanisms can be combined only if the stiffness and deformation capacity of the mechanisms are compatible.

The nominal bearing and shear-friction strengths shall meet the requirements of ACI 318 Chapters 10 and 11. Unless a higher strength is substantiated by cyclic testing, the nominal bearing and shear-friction strengths shall be reduced by 25 percent for the composite seismic systems described in Sections 540, 543, 545, 547, and 548.

2. The available strength of structural steel components in composite connections shall be determined in accordance with Part 2A and the Specification. Structural steel elements that are encased in confined reinforced concrete are permitted to be considered to be braced against out-of-plane buckling. Face bearing plates consisting of stiffeners between the flanges of steel beams are required when beams are embedded in reinforced concrete columns or walls.
3. The nominal shear strength of reinforced-concrete-encased steel panel-zones in beam-to-column connections shall be calculated as the sum of the nominal strengths of the structural steel and confined reinforced concrete shear elements as determined in Part 2A Section 540.3 and ACI 318 Section 21.5, respectively.
4. Reinforcement shall be provided to resist all tensile forces in reinforced concrete components of the connections. Additionally, the concrete shall be confined with transverse reinforcement. All reinforcement shall be fully developed in tension or compression, as appropriate, beyond the point at which it is no longer required to resist the forces. Development lengths shall be determined in accordance with ACI 318 Chapter 12. Additionally, development lengths for the systems described in Sections 540, 543, 545, 547, and 548 shall meet the requirements of ACI 318 Section 21.5.4.
5. Connections shall meet the following additional requirements:
 - a. When the slab transfers horizontal diaphragm forces, the slab reinforcement shall be designed and anchored to carry the in-plane tensile forces at all critical sections in the slab, including connections to collector beams, columns, braces, and walls.
 - b. For connections between structural steel or composite beams and reinforced concrete or encased composite columns, transverse hoop reinforcement shall be provided in the connection region of the column to meet the requirements of ACI 318 Section 21.5, except for the following modifications:

b.1 Structural steel sections framing into the connections are considered to provide confinement over a width equal to that of face bearing plates welded to the beams between the flanges.

b.2 Lap splices are permitted for perimeter ties when confinement of the splice is provided by face bearing plates or other means that prevents spalling of the concrete cover in the systems described in Sections 541, 542, 543 and 546.

b.3 The longitudinal bar sizes and layout in reinforced concrete and composite columns shall be detailed to minimize slippage of the bars through the beam-to-column connection due to high force transfer associated with the change in column moments over the height of the connection.

SECTION 539 COMPOSITE PARTIALLY RESTRAINED (PR) MOMENT FRAMES (C-PRMF)

539.1 Scope

This section is applicable to frames that consist of structural steel columns and composite beams that are connected with partially restrained (PR) moment connections that meet the requirements in Specification Section 502.3.6b (b). Composite partially restrained moment frames (C-PRMF) shall be designed so that under earthquake loading yielding occurs in the ductile components of the composite PR beam-to-column moment connections. Limited yielding is permitted at other locations, such as column base connections. Connection flexibility and composite beam action shall be accounted for in determining the dynamic characteristics, strength and drift of C-PRMF.

539.2 Columns

Structural steel columns shall meet the requirements of Section 519 and 521 and the specification.

539.3 Composite Beams

Composite beams shall be unencased, fully composite and shall meet the requirements of Specification Section 509. For purpose of analysis, the stiffness of the beams shall be determined with an effective moment of inertia of the composite section.

539.4 Moment Connections

The required strength of the beam-to-column PR moment connections shall be determined considering the effects of connection flexibility and second-order moments. In addition, composite connections shall have a nominal strength that is at least equal to 50 percent of M_p , where M_p is the nominal plastic flexural strength of the connected structural steel beam ignoring composite action. Connections shall meet the requirements of Section 520 and shall have a total interstory drift angle of 0.04 radians that is substantiated by cyclic testing as described in Section 522.2b.

SECTION 540 COMPOSITE SPECIAL MOMENT FRAMES (C-SMF)

540.1 Scope

This section is applicable to moment frames that consist of either composite or reinforced concrete columns and either structural steel or composite beams. Composite special moment frames (C-SMF) shall be designed assuming that significant inelastic deformations will occur under the design earthquake, primarily in the beams, but with limited inelastic deformations in the column and/or connections.

540.2 Columns

Composite columns shall meet the requirements for special seismic systems of Sections 537.4 or 537.5, as appropriate. Reinforced concrete columns shall meet the requirements of ACI 318 Chapter 21, excluding Section 21.10.

540.3 Beams

Composite beams that are part of C-SMF shall also meet the following requirements:

1. The distance from the maximum concrete compression fiber to the plastic neutral axis shall not exceed.

$$Y_{PNA} = \frac{Y_{con} + d_b}{1 + \left(\frac{1700F_y}{E} \right)} \quad (540-1)$$

where

- Y_{con} = distance from the top of the steel beam to the top of concrete, mm
 d_b = depth of the steel beam, mm
 F_y = specified minimum yield stress of the steel beam, MPa
 E = elastic modulus of the steel beam, MPa

2. Beam flanges shall meet the requirements of Part 2A Section 540.4, except when reinforced-concrete-encased compression elements have a reinforced concrete cover of at least 50 mm and confinement is provided by hoop reinforcement in regions where plastic hinges are expected to occur under seismic deformations. Hoop reinforcement shall meet the requirements of ACI 318 Section 21.3.3.

Neither structural steel nor composite trusses are permitted as flexural members to resist seismic loads in C-SMF unless it is demonstrated by testing and analysis that the particular system provides adequate ductility and energy dissipation capacity.

540.4 Moment Connections

The required strength of beam-to-column moment connections shall be determined from the shear and flexure associated with the expected flexural strength, $R_y M_n$ (LRFD) or $R_y M_n / 1.5$ (ASD), as appropriate, of the beams framing into the connection. The nominal strength of the connection shall meet the requirements in Section 538. In addition, the connections shall be capable of sustaining a total interstory drift angle of 0.04 radian. When beam flanges are interrupted at the connection, the connections shall demonstrate an interstory drift angle of at least 0.04 radian in cyclic tests that is substantiated by cyclic testing as described in Part 2A Section 540.2.(b). For connections to reinforced concrete columns with a beam that is continuous through the column so that welded joints are not required in the flanges and the connection is not otherwise susceptible to premature fractures, the inelastic rotation capacity shall be demonstrated by testing or other substantiating data.

540.5 Column-Beam Moment Ratio

The design of reinforced concrete columns shall meet the requirements of ACI 318 Section 21.4.2. The column-to-beam moment ratio of composite columns shall meet the requirements of Part 2A Section 522.6 with the following modifications:

1. The available flexural strength of the composite column shall meet the requirements of Specification Section 509 with consideration of the required axial strength, P_{rc} .
2. The force limit for Exception (a) in Part 2A Section 522.6 shall be $P_{rc} < 0.1P_c$.
3. Composite columns exempted by the minimum flexural strength requirement in Part 2A Section 522.6(a) shall have transverse reinforcement that meets the requirements in Section 537.4.3(3).

SECTION 541 COMPOSITE INTERMEDIATE MOMENT FRAMES (C-IMF)

541.1 Scope

This Section is applicable to moment frames that consist of either composite or reinforced concrete columns and either structural steel or composite beams. Composite intermediate moment frames (C-IMF) shall be designed assuming that inelastic deformation under the design earthquake will occur primarily in the beams, but with moderate inelastic deformation in the columns and/or connections.

541.2 Columns

Composite columns shall meet the requirements for intermediate seismic systems of Section 537.4 or 537.5. Reinforced concrete columns shall meet the requirements of ACI 318 Section 21.12.

541.3 Beams

Structural steel and composite beams shall meet the requirements of the Specification.

541.4 Moment Connections

The nominal strength of the connections shall meet the requirements of Section 538. The required strength of beam-to-column connections shall meet one of the following requirements:

1. The required strength of the connection shall be based on the forces associated with plastic hinging of the beams adjacent to the connection.
2. Connections shall meet the requirements of Section 538 and shall demonstrate a total interstory drift angle of at least 0.03 radian in cyclic tests.

SECTION 542 COMPOSITE ORDINARY MOMENT FRAMES (C-OMF)

542.1 Scope

This Section is applicable to moment frames that consist of either composite or reinforced concrete columns and structural steel or composite beams. Composite ordinary moment frames (C-OMF) shall be designed assuming that limited inelastic action will occur under the design earthquake in the beams, columns and/or connections.

542.2 Columns

Composite columns shall meet the requirements for ordinary seismic systems in Section 537.4 or 537.5, as appropriate. Reinforced concrete columns shall meet the requirements of ACI 318, excluding Chapter 21.

542.3 Beams

Structural steel and composite beams shall meet the requirements of the Specification.

542.4 Moment Connections

Connections shall be designed for the load combinations in accordance with Specification Sections 502.3.3 and 502.3.4, and the available strength of the connections shall meet the requirements in Section 520 and Section 524.2 of Part 2A.

SECTION 543 COMPOSITE SPECIAL CONCENTRICALLY BRACED FRAMES (C-CBF)

543.1 Scope

This Section is applicable to braced frames that consist of concentrically connected members. Minor eccentricities are permitted if they are accounted for in the design. Columns shall be structural steel, composite structural steel, or reinforced concrete. Beams and braces shall be either structural steel or composite structural steel. Composite special concentrically braced frames (C-SCBF) shall be designed assuming that inelastic action under the design earthquake will occur primarily through tension yielding and/or buckling of braces.

543.2 Columns

Structural steel columns shall meet the requirements of Part 2A Sections 537 and 539. Composite columns shall meet the requirements for special seismic systems of Section 537.4 or 537.5. Reinforced concrete columns shall meet the requirements for structural truss elements of ACI 318 Chapter 21.

543.3 Beams

Structural steel beams shall meet the requirements for special concentrically braced frames (SCBF) of Part 2A Section 526. Composite beams shall meet the requirements of the Specification Section 509 and the requirements for special concentrically braced frames (SCBF) of Part 2A Section 526.

543.4 Braces

Structural steel braces shall meet the requirements for SCBF of Part 2A Section 526. Composite braces shall meet the requirements for composite columns of Section 543.2.

543.5 Connections

Bracing connections shall meet the requirements of Section 538 and Part 2A Section 526.

SECTION 544

COMPOSITE ORDINARY BRACED FRAMES (C-OBF)

544.1 Scope

This Section is applicable to concentrically braced frame systems that consist of composite or reinforced concrete columns, structural steel or composite beams, and structural steel or composite braces. Composite ordinary braced frames (C-OBF) shall be designed assuming that limited inelastic action under the design earthquake will occur in the beams, columns, braces, and/or connections.

544.2 Columns

Encased composite columns shall meet the requirements for ordinary seismic systems of Sections 537.4. Filled composite columns shall meet the requirements of Section 537.5 for ordinary seismic systems. Reinforced concrete columns shall meet the requirements of ACI 318 excluding Chapter 21.

544.3 Beams

Structural steel and composite beams shall meet the requirements of the Specification.

544.4 Braces

Structural steel braces shall meet the requirements of the Specification. Composite braces shall meet the requirements for composite columns of Sections 537.4a, 537.5, and 544.2.

544.5 Connections

Connections shall be designed for the load combinations in accordance with Specification Sections 502.3.3 and 502.3.4, and the available strength of the connections shall meet the requirements in Section 538.

SECTION 545

COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF)

545.1 Scope

This Section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and column, or intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace. Composite eccentrically braced frames (C-EBF) shall be designed so that inelastic deformations under the design earthquake will occur only as shear yielding in the links.

Diagonal braces, columns, and beam segments outside of the link shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened link. Columns shall be either composite or reinforced concrete. Braces shall be structural steel. Links shall be structural steel as described in this Section. The available strength of members shall meet the requirements in the Specification, except as modified in this Section. C-EBF shall meet the requirements of Part 2A Section 528, except as modified in this Section.

545.2 Columns

Reinforced concrete columns shall meet the requirements for structural truss elements of ACI 318 Chapter 21. Composite columns shall meet the requirements for special seismic systems of Sections 537.4 or 537.5. Additionally, where a link is adjacent to a reinforced concrete column or encased composite column, transverse column reinforcement meeting the requirements of ACI 318 Section 21.4.4 (or Section 537.4c(6) for composite columns) shall be provided above and below the link connection. All columns shall meet the requirements of Part 2A Section 528.10.

545.3 Links

Links shall be unencased structural steel and shall meet the requirement for eccentrically braced frame (EBF) links in Part 2A Section 528. It is permitted to encase the portion of the beam outside of the link in reinforced concrete. Beams containing the link are permitted to act compositely with the floor slab using shear connectors along all or any portion of the beam if the composite action is considered when determining the nominal strength of the link.

545.4 Braces

Structural steel braces shall meet the requirements for EBF of Part 2A Section 528.

545.5 Connections

In addition to the requirements for EBF of Part 2A Section 528, connections shall meet the requirements of Section 520.

SECTION 546

ORDINARY REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS (C-ORCW)

546.1 Scope

The requirements in this Section apply when reinforced concrete walls are composite with structural steel elements, either as infill panels, such as reinforced concrete walls in structural steel frames with unencased or reinforced-concrete encased structural steel sections that act as boundary members, or as structural steel coupling beams that connect two adjacent reinforced concrete walls. Reinforced concrete walls shall meet the requirements of ACI 318 excluding Chapter 21.

546.2 Boundary Members

Boundary members shall meet the requirements of this Section:

1. When unencased structural steel sections function as boundary members in reinforced concrete infill panels, the structural steel sections shall meet the requirements of the Specification. The required axial strength of the boundary member shall be determined assuming that the shear forces are carried by the reinforced concrete wall and the entire gravity and overturning forces are carried by the boundary members in conjunction with the shear wall. The reinforced concrete wall shall meet the requirements of ACI 318 excluding Chapter 21.
2. When reinforced-concrete-encased shapes function as boundary members in reinforced concrete infill panels, the analysis shall be based upon a transformed concrete section using elastic material properties. The wall shall meet the requirements of ACI 318 excluding Chapter 21. When the reinforced-concrete-encased structural steel boundary member qualifies as a composite column as defined in Specification Section 509, it shall be designed to meet the ordinary seismic system requirements of Section 537.4a. Otherwise, it shall be designed as a composite column to meet the requirements of ACI 318 Section 10.16 and Section 509 of the Specification.
3. Headed shear studs or welded reinforcement anchors shall be provided to transfer vertical shear forces between the structural steel and reinforced concrete. Headed shear studs, if used, shall meet the requirements of Specification Section 509. Welded reinforcement anchors, if used, shall meet the

requirements of AWS D1.4.

546.3 Steel Coupling Beams

Structural steel coupling beams that are used between two adjacent reinforced concrete walls shall meet the requirements of the Specification and this Section:

1. Coupling beams shall have an embedment length into the reinforced concrete wall that is sufficient to develop the maximum possible combination of moment and shear that can be generated by the nominal bending and shear strength of the coupling beam. The embedment length shall be considered to begin inside the first layer of confining reinforcement in the wall boundary member. Connection strength for the transfer of loads between the coupling beam and the wall shall meet the requirements of Section 538.
2. Vertical wall reinforcement with nominal axial strength equal to the nominal shear strength of the coupling beam shall be placed over the embedment length of the beam with two-thirds of the steel located over the first half of the embedment length. This wall reinforcement shall extend a distance of at least one tension development length above and below the flanges of the coupling beam. It is permitted to use vertical reinforcement placed for other purposes, such as for vertical boundary members, as part of the required vertical reinforcement.

546.4 Encased Composite Coupling Beams

Encased composite sections serving as coupling beams shall meet the requirements of Section 546.3 as modified in this Section:

1. Coupling beams shall have an embedment length into the reinforced concrete wall that is sufficient to develop the maximum possible combination of moment and shear capacities of the encased composite steel coupling beam.
2. The nominal shear capacity of the encased composite steel coupling beam shall be used to meet the requirement in Section 546.3(1).
3. The stiffness of the encased composite steel coupling beams shall be used for calculating the required strength of the shear wall and coupling beam.

SECTION 547

SPECIAL REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS (C-SRCW)

547.1 Scope

Special reinforced concrete shear walls composite with structural steel elements (C-SRCW) systems shall meet the requirements of Section 15 for C-ORCW and the shear-wall requirement of ACI 318 including Chapter 21, except as modified in this Section.

547.2 Boundary Members

In addition to the requirements of Section 547.2(1), uncased structural steel columns shall meet the requirements of Part 2A Sections 519 and 521.

In addition to the requirements of Section 15.2(2), the requirements in this Section shall apply to walls with reinforced-concrete-encased structural steel boundary members. The wall shall meet the requirements of ACI 318 including Chapter 21. Reinforced-concrete-encased structural steel boundary members that qualify as composite columns in Specification Section 509 shall meet the special seismic system requirements of Section 537.4. Otherwise, such members shall be designed as composite compression members to meet the requirements of ACI 318 Section 10.16 including the special seismic requirements for boundary members in ACI 318 Section 21.7.6. Transverse reinforcement for confinement of the composite boundary member shall extend a distance of $2h$ into the wall, where h is the overall depth of the boundary member in the plane of the wall. Headed shear studs or welded reinforcing bar anchors shall be provided as specified in Section 546.2(3). For connection to uncased structural steel sections, the nominal strength of welded reinforcing bar anchors shall be reduced by 25 percent from their static yield strength.

547.3 Steel Coupling Beams

In addition to the requirements of Section 546.3, structural steel coupling beams shall meet the requirements of Part 2A Sections 528.2 and 528.3. When required in Part 2A Section 528.3, the coupling rotation shall be assumed as 0.08 radian unless a smaller value is justified by rational analysis of the inelastic deformations that are expected under the design earthquake. Face bearing plates shall be provided on both sides of the coupling beams at the face of the reinforced concrete wall. These stiffeners shall meet the detailing requirements of Part 2A Section 528.3.

Vertical wall reinforcement as specified in Section 528.3(2) shall be confined by transverse reinforcement that meets the requirements for boundary members of ACI 318 Section 21.7.6.

547.4 Encased Composite Coupling Beams

Encased composite sections serving as coupling beams shall meet the requirements of Section 546.3, except the requirements of Part 2A Section 528.3 need not be met.

SECTION 548**COMPOSITE STEEL PLATE SHEAR WALLS (C-SPW)****548.1 Scope**

This Section is applicable to structural walls consisting of steel plates with reinforced concrete encasement on one or both sides of the plate and structural steel or composite boundary members.

548.2 Wall Elements

The available shear strength shall be ϕV_{ns} (LRFD) or V_{ns}/Ω (ASD), as appropriate, according to the limit state of shear yielding of composite steel plate shear walls (C-SPW) with a stiffened plate conforming to Section 530.2(1) shall be

$$V_{ns} = 0.6 A_{sp} F_y \quad (548-1)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

$$\begin{aligned} V_{ns} &= \text{nominal shear strength of the steel plate, N} \\ A_{sp} &= \text{horizontal area of stiffened steel plate, mm}^2 \\ F_y &= \text{specified minimum yield stress of the plate, MPa} \end{aligned}$$

The available shear strength of C-SPW with a plate that does not meet the stiffening requirements in Section 530.2(1) shall be based upon the strength of the plate, excluding the strength of the reinforced concrete, and meet the requirements of the Specification Sections 507.2 and 507.3.

1. The steel plate shall be adequately stiffened by encasement or attachment to the reinforced concrete if it can be demonstrated with an elastic plate buckling analysis that the composite wall can resist a nominal shear force equal to V_{ns} . The concrete thickness shall be a minimum of 100 mm on each side when concrete is provided on both sides of the steel plate and 200 mm when concrete is provided on one side of the steel plate. Headed shear stud connectors or other mechanical connectors shall be provided to prevent local buckling and separation of the plate and reinforced concrete. Horizontal and vertical reinforcement shall be provided in the concrete encasement to meet or exceed the detailing requirements in ACI 318 Section 14.3. The reinforcement ratio in both directions shall not be less than 0.0025; the maximum spacing between bars shall not exceed 450 mm.

Seismic forces acting perpendicular to the plane of the wall as specified by this code shall be considered in the design of the composite wall system.

2. The steel plate shall be continuously connected on all edges to structural steel framing and boundary members with welds and/or slip-critical high-strength bolts to develop the nominal shear strength of the plate. The design of welded and bolted connectors shall meet the additional requirements of Part 2A Section 520.

548.3 Boundary Members

Structural steel and composite boundary members shall be designed to resist the shear capacity of plate and any reinforced concrete portions of the wall active at the design story drift. Composite and reinforced concrete boundary members shall also meet the requirements of Section 547.2. Steel boundary members shall also meet the requirements of Part 2A, Section 530.

548.4 Openings

Boundary members shall be provided around openings as required by analysis.

SECTION 549 STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS, SHOP DRAWINGS, AND ERECTION DRAWINGS

Structural design drawings and specifications, shop drawings, and erection drawings for composite steel and steel building construction shall meet the requirements of Part 2A Section 518.

For reinforced concrete and composite steel building construction, the contract documents, shop drawings, and erection drawings shall also indicate the following:

1. Bar placement, cutoffs, lap and mechanical splices, hooks and mechanical anchorages.
2. Tolerance for placement of ties and other transverse reinforcement.
3. Provisions for dimensional changes resulting from temperature changes, creep and shrinkage.
4. Location, magnitude, and sequencing of any prestressing or post-tensioning present.
5. If concrete floor slabs or slabs on grade serve as diaphragms, connection details between the diaphragm and the main lateral-load resisting system shall be clearly identified.

User Note: For reinforced concrete and composite steel building construction, the provisions of the following documents may also apply: ACI 315-04 (Details and Detailing of Concrete Reinforcement), ACI 315R-94 (Manual of Engineering and Placing Drawings for Reinforced Concrete Structures), and ACI SP-66 (ACI Detailing Manual), including modifications required by Chapter 21 of ACI 318-02 and ACI 352 (Monolithic Joints in Concrete Structures).

SECTION 550 QUALITY ASSURANCE PLAN

When required by this code or the engineer-of-record, a quality assurance plan shall be provided. For the steel portion of the construction, the provisions of Part 2A, Section 531 apply.

User Note: For the reinforced concrete portion, the provisions of ACI 121R-98 (Quality Management Systems for Concrete Construction), ACI 309.3R-97 (Guide to Consolidation of Concrete in Congested Areas and Difficult Placing Conditions), ACI 311.1R-01 (ACI Manual of Concrete Inspection) and ACI 311.4R-00 (Guide for Concrete Inspection) may apply.

PART 3 DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

SYMBOLS

A	= Full unreduced cross-sectional area of member
A	= Area of directly connected elements or gross area
A_b	= $b_1t + A_s$, for bearing stiffener at interior support and or under concentrated load, and $b_2t + A_{sr}$ for bearing stiffeners at end support
A_b	= Gross cross-sectional area of bolt
A_c	= $18t^2 + A_{s1}$, for bearing stiffener at interior support or under concentrated load, and $10t^2 + A_{s1}$, for bearing stiffeners at end support
A_e	= Effective area at stress F_n
A_e	= Effective net area
A_f	= Cross-sectional area of compression flange plus edge stiffener
A_g	= Gross area of element including stiffeners
A_g	= Gross area of section
A_{gv}	= Gross area subject to shear
A_{nt}	= Net area subject to tension
A_{nv}	= Net area subject to shear
A_n	= Net area of cross-section
A_d	= Reduced-area due to local buckling
A_p	= Gross-sectional area of roof panel per unit width
A_s	= Cross-sectional area of bearing stiffener
A_s	= Gross area of stiffener
A_{st}	= Gross area of shear stiffener
A_t	= Net tensile area
A_w	= Area of web
A_{wn}	= Net web area
a	= Shear panel length of unreinforced web element, or distance between shear stiffeners of reinforced web elements
a	= Intermediate fastener or spot weld spacing
a	= Fastener distance from or outside web edge
a	= Length of bracing interval
B_c	= Term for determining tensile yield point of corners
b	= Effective design width of compression element
b	= Flange width
b_d	= Effective width for deflection calculation

b_e	= Effective width of elements, located at centroid of element including stiffeners	D	= Dead load
b_e	= Effective width	D_2, D_3	= Lip dimension
b_e	= Effective width determined either by section 552.4 or Section 552.5.1 depending on stiffness of stiffeners	d	= Depth of section
b_o	= Total flat width of stiffened element	d	= Nominal screw diameter
b_o	= Total flat width of edge stiffened element	d	= Flat depth of lip defined in Figure 552-9
b_p	= Largest sub-element flat width	d	= Width of arc seam weld
$b_1 b_2$	= Effective widths	d	= Visible diameter of outer surface of arc spot weld
$b_1 b_2$	= Effective widths of bearing stiffeners	d	= Diameter of bolt
C	= For compression members, ratio of total corner cross-sectional area to total cross-sectional area of full section; for flexural members, ratio of total corner cross-sectional area of controlling flange to full cross-sectional area of controlling flange	d_a	= Average diameter of arc spot weld at mid-thickness of t
C	= Coefficient	d_a	= Average width of seam weld
C	= Bearing factor	d_b	= Nominal diameter (body or shank diameter)
C_b	= Bending coefficient dependent on moment gradient	d_e	= Effective diameter of fused area
C_f	= Constant from Table 557.1	d_e	= Effective width of arc seam weld at fused surfaces
C_h	= Web slenderness coefficient	d_h	= Diameter of hole
C_m	= End moment coefficient in interaction formula	d_h	= Depth of hole
C_{mx}	= End moment coefficient in interaction formula	d_h	= Diameter of standard hole
C_{my}	= End moment coefficient in interaction formula	$d_{pi,j}$	= Distance along roof slope between the i^{th} purlin line and the j^{th} anchorage device
C_N	= Bearing length coefficient	d_s	= Reduced effective width of stiffener
C_p	= Correction factor	d'_s	= Depth of stiffener
C_R	= Inside bend radius coefficient	d'_s	= Effective width of stiffener calculated according to Section 552.3
C_s	= Coefficient for lateral-torsional buckling	d_{wx}	= Screw head or washer diameter
C_{TF}	= End moment coefficient in interaction formula	d_w	= Larger value of screw head or washer diameter
C_v	= Shear stiffener coefficient	E	= Modulus of elasticity of steel, 203,000 Mpa, or 2,070,000 kg/cm ²
C_w	= Torsional warping constant of cross-section	E	= Live load due to earthquake
C_{wf}	= Torsional warping constant of flange	E	= Twist of stud from initial, ideal, unbuckled shape
C_y	= Compression strain factor	E^*	= Reduced modulus of elasticity for flexural and axial stiffness in second-order analysis
C_1, C_2, C_3	= Axial buckling coefficients	e	= Distance measured in line of force from center of a standard hole to nearest edge of an adjacent hole or to end of connected part toward which the force is directed
C_1 to C_6	= Coefficients tabulated in Tables 554-3 to 554-5	e	= Distance measured in line of force from center of a standard hole to nearest end of connected part
C_ϕ	= Calibration coefficient	e_{min}	= Minimum allowable distance measured in line of force from centerline of a weld to nearest edge of an adjacent weld or to end of connected part toward which the force is directed
c	= Strip of flat width adjacent to hole	e_{sx}, e_{sy}	= Eccentricities of load components measured from the shear center and in the x and y directions, respectively
c	= Distance	e_y	= Yield strain = F_y/E
c_f	= Amount of curling displacement	F	= Fabrication factor
c_i	= Horizontal distance from edge of element to centerline of stiffener	F_{SR}	= Design stress range
D	= Outside diameter of cylindrical tube		
D	= Overall depth of lip		
D	= Shear stiffener coefficient		

F_{TH}	= Threshold fatigue stress range	f_d	= Computed compressive stress in element being considered. Calculations are based on effective section at load for which deflections are determined
F_c	= Critical buckling stress	f_{d1}, f_{d2}	= Computed stresses f_1 and f_2 in unstiffened element, as defined in Figures 552-5 to 552-8. Calculations are based on effective section at load for which serviceability is determined
F_{cr}	= Plate elastic buckling stress	f_v	= Required shear stress on a bolt
F_d	= Elastic distortional buckling stress	f_1, f_2	= Stresses on unstiffened element defined by Figures 552-6 to 552-8
F_e	= Elastic buckling stress	f_1, f_2	= Stresses at the opposite ends of web
F_m	= Mean value of fabrication factor	G	= Shear modulus of steel, 78,000 Mpa or 795,000 kg/cm ²
F_n	= Nominal buckling stress	g	= Vertical distance between two rows of connections nearest to top and bottom flanges
F_n	= Nominal strength resistance of bolts	g	= Transverse center-to-center spacing between fastener gage lines
F_{nt}	= Nominal tensile strength resistance of bolts	g	= Gauge, spacing of fastener perpendicular to force
F_{nv}	= Nominal shear strength resistance of bolts	H	= A permanent load due to lateral earth pressure, including groundwater
F'_{nt}	= Nominal tensile strength resistance for bolts subject to combination of shear and tension	h	= Depth of flat portion of web measured along plane of web
F_{sy}	= Yield stress as specified in Section 551.2.1, 551.2.2, 551.2.3	h	= Width of elements adjoining stiffened element
F_t	= Nominal tensile stress in flat sheet	h	= Lip height as defined in Figures 555-15 to 555-18
F_u	= Tensile strength as specified in Section 551.1.1, 551.2.2, or 551.2.3	h_o	= Overall depth of unstiffened C-section member as defined in Figure 552-8
F_{uv}	= Tensile strength of virgin steel specified by Section 551.2 or established in accordance with Section 556.3.3	h_s	= Depth of soil supported by the structure
F_{wy}	= Lower value of F_y for beam web or F_{ys} for bearing stiffeners	h_{wc}	= Coped flat web depth
F_{xx}	= Tensile strength of electrode classification	h_x	= x distance from the centroid of flange to the flange / web junction
F_{u1}	= Tensile strength of members in contact with screw head	I_E	= Importance factor for earthquake
F_{u2}	= Tensile strength of member not in contact with screw head	I_W	= Importance factor for wind
F_v	= Nominal shear stress	I_a	= Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element
F_y	= Yield stress used for design, not to exceed specified yield stress or established in accordance with Section 556, or as increased for cold work of forming in Section 551.7.2 or as reduced for low ductility steels in section	I_{eff}	= Effective moment of inertia
F_{ya}	= Average yield stress of section	I_g	= Gross moment of inertia
F_{yc}	= Tensile yield stress of corners	I_s	= Actual moment of inertia of full stiffener about its own centroidal axis parallel to element to be stiffened
F_{yf}	= Weighted average tensile yield stress of flat portions	I_{smin}	= Minimum moment of inertia of shear stiffener(s) with respect to an axis in plane of web
F_{ys}	= Yield stress of stiffener steel	I_{sp}	= Moment of inertia of stiffener about centerline of flat portion of element
F_{yv}	= Tensile yield stress of virgin steel specified by Section 551.2 or established in accordance with Section 556.3.3	I_x, I_y	= Moment of inertia of full unreduced section about principal axis
f	= Stress in compression element computed on basis of effective design width	I_{xf}	= x-axis moment of inertia of the flange
f_{av}	= Average computed stress in full unreduced flange width	I_{xy}	= Product of inertia of full unreduced section about major and minor centroidal axes
f_c	= Stress at service load in cover plate or sheet		
$f_{bending}$	= Normal stress due to bending alone at the maximum normal on the cross section due to combined bending and torsion		
$f_{torsion}$	= Normal stress due to torsion alone at the maximum normal stress on the cross section due to combined bending and torsion		

I_{xyf}	= Product of inertia of flange about major and minor centroidal axes	L	= Length of longitudinal welds
I_{yc}	= Moment of inertia of compression portion of section about centroidal axis of entire section parallel to web using unreduced section	L	= Length of seam weld not including circular ends
I_{yf}	= y-axis moment of inertia of flange	L	= Length of connection
i	= Index of stiffener	L	= Unbraced length of member
i	= Index of each purlin line	L	= Overall length
J	= Saint-Venant torsion constant	L	= Live load
J_f	= Saint-Venant torsion constant of compression flange plus edge stiffener about an x-y axis located at the centroid of the flange	L	= Minimum of L_{cr} and L_m
j	= Section property for torsional-flexural buckling	L_b	= Distance between braces on one compression member
j	= Index for each anchorage device	L_{br}	= Unsupported length between brace points or other restraints which restrict distortional buckling of element
K	= Effective length factor	L_c	= Summation of critical path lengths of each segment
K'	= A constant	L_{cr}	= Critical unbraced length of distortional buckling
K_a	= Lateral stiffness of anchorage device	L_{gv}	= Gross failure path length parallel to force
K_{af}	= Parameter for determining axial strength of Z-Section member having one flange fastened to sheathing	L_h	= Length of hole
$K_{effi,j}$	= Effective lateral stiffness of j^{th} anchorage device with respect to i^{th} purlin	L_m	= Distance between discrete restraints that restrict distortional buckling
K_{req}	= Required stiffness	L_{nv}	= Net failure path length parallel to force
K_{sys}	= Lateral stiffness of roof system, neglecting anchorage devices	L_o	= Overhang length measured from the edge of bearing to the end of member
K_t	= Effective length factor for torsion	L_o	= Length at which local buckling stress equals flexural buckling stress
K_{totali}	= Effective lateral stiffness of all elements resisting force	L_s	= Net failure path length inclined to force
K_x	= Effective length factor for buckling about x-axis	L_{st}	= Length of bearing stiffener
K_y	= Effective length factor for buckling about y-axis	L_t	= Unbraced length of compression member for torsion
k	= Plate buckling coefficient	L_t	= Net failure path length normal to force due to direct tension
k_d	= Plate buckling coefficient for distortional buckling	L_u	= Limit of unbraced length below which lateral-torsional buckling is not considered
k_{loc}	= Plate buckling coefficient for local sub-element buckling	L_x	= Unbraced length of compression member for bending about x-axis
k_v	= Shear buckling coefficient	L_y	= Unbraced length of compression member for bending about y-axis
k_ϕ	= Rotational stiffness	l	= Distance from concentrated load to a brace
$k_{\phi fe}$	= Elastic rotational stiffness provided by the flange to the flange/web juncture	M	= Required allowable flexural strength, ASD
$\bar{k}_{\phi fg}$	= Geometric rotational stiffness demanded by the flange from the flange/web juncture	M	= Bending moment
$k_{\phi we}$	= Elastic rotational stiffness provided by the web to the flange/web juncture	M_{crd}	= Distortional buckling moment
$\bar{k}_{\phi wg}$	= Geometric rotational stiffness demanded by the web from the flange/web juncture	M_{cre}	= Overall buckling moment
L	= Full span for simple beams, distance between inflection point for continuous beams, twice member length for cantilever beams	M_{crl}	= Local buckling moment
L	= Span length	M_d	= Nominal moment with consideration of deflection
L	= Length of weld	M_f	= Factored moment
		M_{fx}, M_{fy}	= Moments due to factored loads with respect to centroidal axes
		M_m	= Mean value of material factor
		M_{max}	= Absolute value of moment in unbraced segment
		M_A, M_B	= used for determining C_b
		M_C	

- M_n = Nominal flexural strength
 M_{nd} = Nominal flexural strength for distortional buckling
 M_{ne} = Nominal flexural strength for overall buckling
 M_{nl} = Nominal flexural strength for local buckling
 M_{nx} = Nominal flexural strengths about Section 553.
 M_{ny} = Nominal flexural strengths [resistance] about centroidal axes determined in accordance with Section 553.1 excluding provisions of Section 553.1.1.b
 M_{nxt} = Nominal flexural strength about centroidal axes determined using gross, unreduced cross-section properties
 M_{nyt} = Required allowable flexural strength with respect to centroidal axes for ASD
 M_x, M_y = Required flexural strength with respect to centroidal axes for LRFD
 M_u = Required flexural strength with respect to Centroidal axes for LRFD
 M_{uy} = moment causing maximum strain e_y
 M_y = Yield moment ($= S_f F_y$)
 M_1 = Smaller end moment in an unbraced segment
 M_2 = Larger end moment in an unbraced segment
 $\bar{M}$ = Required flexural strength factored moment
 $\bar{M}_x, \bar{M}_y$ = Required flexural strength factored moment
 M_z = Torsional moment of required load P about shear center
 m = Degrees of freedom
 m = Term for determining tensile yield point of corners
 m = Distance from shear center of one C-section to mid-plane of web
 m_f = Modification factor for type of bearing connection
 N = Actual length of bearing
 N = Number of stress range fluctuations in design life
 N_a = Number of anchorage devices along a line of anchorage
 N_i = Notional lateral load applied at level i
 N_p = Number of purlin lines on roof slope
 n = Coefficient
 n = Number of stiffeners
 n = Number of holes
 n = Number of tests
 n = Number of equally spaced intermediate brace locations
 n = Number of anchors in test assembly with same tributary area (for anchor failure), or number of panels with identical spans and loading to failed span (for non-anchor failure)
 n = Number of threads per inch
- n_b = Number of bolt holes
 n_c = Number of compression flange stiffeners
 n_w = Number of web stiffeners and/or folds
 n_t = Number of tension flange stiffeners
 P = Required allowable strength for concentrated load reaction in presence of bending moment for ASD
 P = Required allowable strength (nominal force) transmitted by weld for ASD
 P = Required allowable compressive axial strength for ASD
 P = Professional factor
 P = Required concentrated load (factored load) within a distance of $0.3a$ on each side of a brace, plus $1.4(1 - l/a)$ times each required concentrated load located farther than $0.3a$ but not farther than $1.0a$ from the brace
 P = Required nominal brace strength resistance for a single compression member
 P_{Ex}, P_{Ey} = Elastic buckling strengths resistance
 P_{L1}, P_{L2} = Lateral bracing forces
 P_{Lj} = Lateral force to be resisted by the j^{th} anchorage device
 P_{crd} = Distortional buckling load
 P_{crl} = Local buckling load
 P_{ere} = Overall buckling load
 P_f = Axial force due to factored loads
 P_f = Concentrated load or reaction due to factored loads
 P_i = Lateral force introduced into the system at the i^{th} purlin
 P_m = Mean value of the tested-to-predicted load ratios
 P_n = Nominal web crippling strength
 P_n = Nominal axial strength resistance of member
 P_n = Nominal axial strength resistance of bearing stiffener
 P_n = Nominal strength resistance of connection component
 P_n = Nominal bearing strength
 P_n = Nominal tensile strength resistance of welded member
 P_n = Nominal bolt strength
 P_{nc} = Nominal web crippling strength resistance of C- or Z-Section with overhang(s)
 P_{nd} = Nominal axial strength for distortional buckling
 P_{ne} = Nominal axial strength for overall buckling
 P_{nl} = Nominal axial strength for local buckling
 P_{no} = Nominal axial strength resistance of member determined in accordance with Section 553.3.4 with $F_n = F_y$

P_{not}	= Nominal pull-out strength resistance per screw	R_n	= Nominal strength
P_{nov}	= Nominal pull-over strength resistance per screw	R_n	= Nominal block shear rupture strength
P_{ns}	= Nominal shear strength resistance per screw	R_n	= Average value of all test results
P_{nt}	= Nominal tension strength resistance per screw	R_r	= Reduction factor
P_r	= Required axial compressive strength resistance	R_u	= Required strength for LRFD
P_s	= Concentrated load or reaction	r	= Correction factor
P_{ss}	= Nominal shear strength resistance of screw as reported by manufacturer or determined by independent laboratory testing	r	= Least radius of gyration of full unreduced cross-section.
P_{ts}	= Nominal tension strength [resistance] of screw as reported by manufacturer or determined by independent laboratory testing	r	= Centerline bend radius.
P_u	= Required axial strength for LRFD	r_i	= Minimum radius of gyration of full unreduced cross-section
P_u	= Factored force required strength transmitted by weld, for LRFD	r_o	= Polar radius of gyration of cross-section about shear center
P_u	= Required strength for concentrated load reaction in presence of bending moment for LRFD	r_x, r_y	= Radius of gyration of cross-section about centroidal principal axis
P_{wc}	= Nominal web crippling strength resistance for C-Section flexural member	S	= $1.28 \sqrt{E/f}$
$P_x P_y$	= Components of required load P parallel to x and y axis, respectively	S	= Variable load due to snow, including ice and associated rain or rain
P_y	= Member yield strength	S_c	= Elastic section modulus of effective section calculated relative to extreme compression fiber at F_c
$\bar{P}$	= Required strength for concentrated load or reaction concentrated load reaction due to factored loads in presence of bending moment	S_e	= Elastic section modulus of effective section calculated relative to extreme compression or tension fiber at F_y
$\bar{P}$	= Required compressive axial strength	S_f	= Elastic section modulus of full unreduced section relative to extreme compression fiber
p	= Pitch (mm per thread for SI units and cm per thread for MKS units)	S_{ft}	= Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis
Q	= Required allowable shear strength of connection	S_{fy}	= Elastic section modulus of full unreduced section relative to extreme fiber in first yield
$\bar{Q}$	= Required shear strength (factored shear force) of connection	S_n	= In-plane diaphragm nominal shear strength resistance
Q_i	= Load effect	s	= Center-to-center hole spacing
q	= Design load in plane of web	s	= Spacing in line of stress of welds, rivets, or bolts connecting a compression cover plate or sheet to a non-integral stiffener or other element
q_s	= Reduction factor	s	= Sheet width divided by number of bolt holes in cross-section being analyzed
R	= Required allowable strength for ASD	s	= Weld spacing
R	= Modification factor	s	= Pitch, spacing of fastener parallel to force
R	= Reduction factor	s'	= Longitudinal center-to-center spacing of any consecutive holes
R	= Reduction factor determined from uplift tests in accordance with AISI S908	s_{end}	= Clear distance from the hole at ends of member
R	= Coefficient	s_{max}	= Maximum permissible longitudinal spacing of welds or other connectors joining two C-sections to form an I-section
R	= Inside bend radius	T	= Required allowable tensile axial strength for ASD
R	= Radius of outside bend surface	T	= Required allowable tension strength of connection
R_I	= $I_s I_a \leq 1$	T	= Load due to contraction or expansion caused by temperature changes
R_a	= Allowable design strength		
R_b	= Reduction factor		
R_c	= Reduction factor		
R_f	= Effect of factored loads		

T_f	= Tension due to factored loads for LSD
T_f	= Factored tensile force of connection for LSD
T_s	= Design strength factored resistance connection in tension
T_u	= Required tensile axial strength for LRFD
T_u	= Required tension strength of connection for LRFD
$\bar{T}$	= Required tensile axial strength factored tensile force
$\bar{T}$	= Required tension strength of connection factored tensile force
t	= Base steel thickness of any element or section
t	= Thickness of coped web
t	= Total thickness of two welded sheets
t	= Thickness of thinnest connected part
t_c	= Lesser of depth of penetration and t_2
t_e	= Effective throat dimension of groove weld
t_i	= Thickness of uncompressed glass fiber blanket insulation
t_s	= Thickness of stiffener
t_w	= Effective throat of weld
t_1, t_2	= Based thickness connected with fillet weld
t_1	= Thickness of member in contact with screw head
t_2	= Thickness of member not in contact with screw head
U	= Reduction coefficient
V	= Required allowable strength for ASD
V_F	= Coefficient of variation of fabrication factor
V_f	= Shear force due to factored loads for LSD
V_f	= Factored shear force of connection for LSD
V_M	= Coefficient of variation of material factor
V_n	= Nominal shear strength
V_p	= Coefficient of variation of tested-to-predicted load ratios
V_Q	= Coefficient of variation of load effect
V_u	= Required shear strength for LRFD
V_u	= Required shear strength of connection for LRFD
$\bar{V}$	= Required shear strength factored shear
W	= Wind load, a variable load due to wind
W	= Required strength from critical load combinations for ASD, LRFD, or LSD
W_{pi}	= Total required vertical load supported by i^{th} purlin in a single bay
W_x, W_y	= Components of required strength W
w	= Flat width of element exclusive of radii
w	= Flat width of beam flange which contacts bearing plate
w	= Flat width of narrowest unstiffened compression element tributary to connections

w_f	= Width of flange projection beyond web for I-beams and similar sections; or half distance between webs for box or U-type sections
w_i	= Required distributed gravity load supported by the i^{th} purlin per unit length
w_o	= Out-to-out width
w_1	= Leg of weld
w_2	= Leg of weld
x	= Non-dimensional fastener location
x	= Nearest distance between web hole and edge of bearing
x_o	= Distance from shear center to centroid along principal x-axis
x_o	= Distance from flange/web junction to the centroid of the flange
x_o	= Distance from flange or web junction to the centroid of the flange
$\bar{x}$	= Distance from shear plane to centroid of cross-section
Y	= Yield point of web steel divided by yield point of stiffener steel
Y_i	= Gravity load from the LRFD or 1.6 times the ASD load combinations applied at level i
y_o	= y distance from centroid of flange to shear center of flange
a	= Coefficient for purlin directions
a	= Coefficient for conversion of units
a	= Load factor
a	= Coefficient for strength resistance increase due to overhang
a	= Coefficient accounts for the benefit of an unbraced length, L_m , shorter than L_{cr}
$l/a_x, l/a_y$	= Magnification factors
β	= Coefficient
β	= A value accounting for moment gradient
$\beta_{br,1}$	= Required brace stiffness for a single compression member
β_o	= Target reliability index
Δ_{tf}	= Lateral displacement of purlin top flange at the line restraint
$\delta, \delta_i, \gamma, \gamma_i, \omega, \omega_i$	= Coefficient
ξ_{web}	= Stress gradient in web
γ_i	= Load factor
θ	= Angle between web and bearing surface > 45° but no more than 90°
θ	= Angle between vertical and plane of web of Z-section, degrees
θ_2, θ_3	= Angle of segment of complex lip
λ, λ_c	= Slenderness factors
λ_1, λ_2	= Parameters used in determining compression strain Factor
λ_3, λ_4	= Slenderness factor
λ_ℓ	= Slenderness factor
λ_d	= Slenderness factor

μ	=	Poisson's ration for steel = 0.30
ρ	=	Reduction factor
σ_{ex}	=	$(\pi^2 E)/(K_x L_x/r_x)^2$
	=	$(\pi^2 E)/(L_x/r_x)^2$
σ_{ey}	=	$(\pi^2 E)/(K_y L_y/r_y)^2$
	=	$(\pi^2 E)/(L_y/r_y)^2$
σ_t	=	Torsional buckling stress
ϕ	=	Resistance factor
ϕ_b	=	Resistance factor bending
ϕ_c	=	Resistance factor for concentrically loaded compression strength
ϕ_d	=	Resistance factor for diaphragms
ϕ_t	=	Resistance factor for tensile strength
ϕ_u	=	Resistance factor for fracture on net section
ϕ_v	=	Resistance factor for shear strength
ϕ_w	=	Resistance factor for web crippling strength
ψ	=	$ f_2/f_1 $
τ_b	=	Parameter for reduced stiffness using second-order analysis
Ω	=	Safety factor
Ω_b	=	Safety factor for bending strength
Ω_c	=	Safety factor for concentrically loaded compression strength
Ω_d	=	Safety factor for diaphragms
Ω_t	=	Safety factor for tension strength
Ω_v	=	Safety factor for shear strength
Ω_w	=	Safety factor for web crippling strength

SECTION 551

GENERAL PROVISIONS

551.1 Scope, Applicability and Definitions

551.1.1 Scope

This specification applies to the design of structural members cold-formed to shape from carbon or low-alloy steel sheet, strip, plate, or bar not more than 25 mm in thickness and used for load-carrying purposes in

1. Buildings; and
2. Structures other than buildings provided allowances are made for dynamic effects.

551.1.2 Applicability

This Specification includes Symbols and Definitions, Section 551 through Section 557, Section C-1, to Section C-3 that shall apply as follows:

Section C-1	Design of Cold-Formed Steel Structural Members Using Direct Design Strength Method
Section C-2	Second-Order analysis.
Section C-3	Additional Provisions

This Specification includes design provisions for

1. Allowable Strength Design (ASD), and
2. Load and Resistance Factor Design (LRFD).

The nominal strength and stiffness of cold-formed steel elements, members, assemblies, connections, and details shall be determined in accordance with the provisions in Section 552 to Section 557, Section C-1 to Section C-3 of the Specification.

Where the composition or configuration of such components is such that calculation of strength and/or stiffness cannot be made in accordance with those provisions, structural performance shall be established from either of the following:

1. Available strength or stiffness by tests, undertaken and evaluated in accordance with Section 556,
2. Available strength or stiffness by rational engineering analysis based on appropriate theory, related testing if data is available, and engineering judgment. Specifically, the available strength is determined from the calculated nominal strength by applying the following safety factors or resistance factors:

For Members

$$\Omega = 2.00 \text{ (ASD)} \quad \phi = 0.80 \text{ (LRFD)}$$

For Connections

$$\Omega = 2.50 \text{ (ASD)} \quad \phi = 0.65 \text{ (LRFD)}$$

551.1.3 Definitions

In this Specification, “shall” is used to express a mandatory requirement, i.e., a provisions that the user is obliged to satisfy in order to comply with the Specification; and “shall be permitted” is used to express an option or that which is permissible within the limits of the Specification.

551.1.4 Units of Symbols and Terms

The unit systems considered in those sections is SI units.

551.2 Material

551.2.1 Applicable Steels

This Specification requires the use of steels intended for structural applications as defined in general by the specifications of the American Society for Testing Materials listed in this section. The term SS shall designate sheet material and the terms HSLAS and HSLAS-F shall designate high-strength low-alloy steels.

ASTM A36/A36M, Standard Specification for Carbon Structural Steel

ASTM A242/A242M, Standard Specification for High-Strength Low-Alloy Structural Steel

ASTM A283/A283M, Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates

ASTM A500, Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M, Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality

ASTM A572/A572M, Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel

ASTM A588/A588M, Standard Specification for High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100mm thick

ASTM A606, Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A 653M/A653M (SS 230 MPa, 255 MPa, 275 MPa, 340 MPa Class 1, Class 3 and Class 4, and 380 MPa; HSLAS and HSLAS-F, 275 MPa, 340 MPa, 380 MPa Class 1 and 2, 410 MPa, & 480 MPa and 550 MPa, Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

ASTM A792/A792M. (230 MPa, 255 MPa, 275 MPa, and 340 MPa Class 1 and Class 4), Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process.

ASTM A847/A847M, Standard Specification for Cold-Formed Welded and Seamless High Strength, Low Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance

ASTM A875/A875M (SS 230 MPa, 255 MPa, 275 MPa, and 340 MPa Class 1 and 3; HSLAS and HSLAS-F, 340 MPa, 410 MPa, 480 MPa, and 550 MPa), Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process

ASTM A1003/A1003M (ST 340 MPa H, 275 MPa H, 255 MPa H, 230 MPa H), Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members

ASTM A1008/A1008M (SS 170 MPa, 205 MPa, 230 MPa Types 1 and 2, and 275 MPa Types 1 and 2; HSLAS Classes 1 and 2, 310 MPa, 340 MPa, 380 MPa, 410 MPa, 450 MPa, and 480 MPa; HSLAS-F 340 MPa, 410 MPa, 480 MPa, and 550 MPa), Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable

ASTM A1011/A1011M (SS 205 MPa, 230 MPa, 250 MPa Types 1 and 2, 275 MPa, 310 MPa, 340 MPa, and 380 MPa; HSLAS Classes 1 and 2, 310 MPa, 340 MPa, 380 MPa, 410 MPa, 450 MPa, and 480 MPa; HSLAS-F 340MPa, 410 MPa, 480 MPa, and 550MPa), Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability

ASTM A1039/A1039M (SS 275 MPa, 340 MPa, 380MPa, 410 MPa, 480 MPa, and 550 MPa), Standard Specification for Steel, Sheet, Hot-Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll Casting Process. Thicknesses of 380 MPa and higher that do not meet the minimum 10% elongation requirement are limited per Section 551.2.3.2.

551.2.2 Other Steels

See Section 551.2.2 of Section C-3

551.2.3 Ductility

Steels not listed in Section 551.2.1 and used for structural members and connections in accordance with Section 551.2.2 shall comply with ductility requirements in either Section 551.2.3.1 or Section 551.2.3.2:

551.2.3.1 General

The ratio of tensile strength to yield stress shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a 50 mm gauge length or 7 percent for a 200 mm gauge length standard specimen tested in accordance with ASTM A370. If these requirements cannot be met, the following criteria shall be satisfied:

1. Local elongation in a 12.7 mm gauge length across the fracture shall not be less than 20 percent, and
2. Uniform elongation outside the fracture shall not be less than 3 percent. When material ductility is determined on the basis of the local and uniform elongation criteria, the use of such material shall be restricted to the design of purlins, girts, and curtain wall studs in accordance with Sections 553.3.1 (a), Section 553.3.2, Section 554.6.1, Section 554.6.2, Section 554.6.2a, and requirements given in Section C-3.2.1 of the Section C-3. For purlins, girts, and curtain wall studs subject to combined axial load and bending moment (Section 553.3.5) $\Omega_c P/P_n$ shall not exceed 0.15 for ASD, $P_u/\phi_c P_n$ shall not exceed 0.15 for LRFD, $P_f/\phi_c P_n$ shall not exceed 0.15 for LSD.

551.2.3.2 Steels

Steels conforming to ASTM A653/A653M SS (550 MPa), A1008/ A1008M SS (550 Mpa), A792/A792M (550 Mpa), A875/ A875M SS (550 Mpa), thicknesses of ASTM A1039 Grades (380Mpa), (410 MPa) , (480 MPa), and (550MPa) that do not meet the minimum 10 percent elongation requirement in Section 551.2.3.1, and other steels that do not meet the provisions of Section 551.2.3.1 shall be permitted for concentrically loaded closed box section compression members as given in Exception 2 below and for multiple-web configurations such as roofing, siding, and floor decking as given in Exception 1 provided that:

1. The yield stress, F_y , used for determining nominal strength nominal resistance in Sections 551 to Section 555 is taken as 75 percent of the specified minimum yield stress or 410 MPa, whichever is less, and
2. The tensile strength, F_u , used for determining nominal strength nominal resistance in Section 555 is taken as 75 percent of the specified minimum tensile strength or 427 MPa, whichever is less.

Alternatively, the suitability of such steels for any multi-web configuration shall be demonstrated by loads tests in accordance with the provisions of Section 556. Available strengths based on these tests shall not exceed the available strengths calculated in accordance with Section 552 through Section 557, Section C-1 to Section C-3, using the specified minimum yield stress, F_{sy} , and the specified minimum tensile strength, F_u .

Exception 1:

For multiple-web configurations, a reduced specified minimum yield stress, $R_b F_{sy}$, shall be permitted for determining the nominal flexural strength in Section 553.3.1a, for which the reduction factor, R_b , shall be determined in accordance with (a) or (b):

- a. For stiffened and partially stiffened compression flanges

$$\text{For } w/t \leq 0.067E / F_{sy} \quad (551.2-1)$$

$$R_b = 1.0$$

$$\text{For } 0.067E / F_{sy} < w/t < 0.974E / F_{sy}$$

$$R_b = 1 - 0.26[(wF_{sy}/(tE)) - 0.067]^{0.4}$$

$$\text{For } 0.974E / F_{sy} \leq w/t \leq 500$$

$$R_b = 0.75$$

b. For unstiffened compression flanges

$$\text{For } w/t \leq 0.0173E/F_{sy} \quad (551.2-2)$$

$$R_b = 1.0$$

$$\text{For } 0.0173E/F_{sy} < w/t \leq 60$$

$$R_b = 1.079 - 0.6 \sqrt{\frac{wF_{sy}}{tE}}$$

where

- w = Flat width of compression flange
- t = Thickness of section
- E = Modulus of elasticity of steel
- F_{sy} = Specified minimum yield stress determined in accordance with Section 551.6 ≤ 550 MPa

The above exception shall not apply to the use of steel deck for composite slabs, for which the steel deck acts as the tensile reinforcement of slab.

Exception 2:

For concentrically loaded compression members with a closed box section, a reduced yield stress, $0.9F_{sy}$, shall be permitted to be used in place of F_y in Eqs. 553.4.2, 553.4.3, and 553.4.4 for determining the axial strength in Section 553.4. A reduced radius of gyration (R_r)(r) shall be used in Eq. 553.4.1 when the value of the effective length KL is less than $1.1L_0$ is given by Eq. 551-3, and R_r is given by Eq. 551-4.

$$L_0 = \pi r \sqrt{\frac{E}{F_{cr}}} \quad (551.2-3)$$

$$R_r = 0.65 + \frac{0.35(KL)}{1.1L_0} \quad (551.2-4)$$

where

- L_0 = Length at which local buckling stress equals flexural buckling stress
- r = Radius gyration of full unreduced cross section
- F_{cr} = Minimum critical buckling stress for section calculated by Eq. 552.2
- R_{cr} = Reduction factor
- KL = Effective length

551.2.4 Delivered Minimum Thickness

The uncoated minimum steel thickness of the cold-formed steel product as delivered to the job site shall not at any location be less than 95 percent of the thickness, t , used in its design; however, lesser thicknesses shall be permitted at bends, such as corners, due to cold-forming effects.

551.3 Loads

Loads and load combinations shall be as stipulated by the applicable provisions in Section C-3.3 of Section C-3.

551.4 Allowable Strength Design

551.4.1 Design Basis

Design under this section of the Specification shall be based on Allowable Strength Design (ASD) principles. All provisions of this Specification shall apply, except for those in Sections 551.5 and in Section 553 and Section 556 designated for LRFD.

551.4.1.1 ASD Requirements

A design satisfies the requirements of this Specification when the allowable strength of each structural component equals or exceeds the required strength, determined on the basis of the nominal loads, for all applicable load combinations.

The design shall be performed in accordance with Eq. 551.4.1-1:

$$R \leq R_n/\Omega \quad (551.4-1)$$

where

- R = Required strength
- R_n = Nominal Strength specified in Section 552 through Section 557 and section C-1
- Ω = Safety factor specified in Section 552 through Section 557 and section C-1
- R_n/Ω = Allowable strength

551.4.1.2 Load Combinations for ASD

Load combination for ASD shall be as stipulated by Section C-3.3.1.1a of Section C-3.

551.5 Load and Resistance Factor Design

551.5.1 Design Basis

Design under this section of the Specification shall be based on Load and Resistance Factor Design (LRFD) principles. All provisions of this Specification shall apply except for those in Sections 551.4 and in Chapters 553 and 556 designated for ASD and LRFD.

551.5.1.1 LRFD Requirements

A design satisfies the requirements of this Specification when the design strength of each structural component equals or exceeds the required strength determined on the basis of the nominal loads, multiplied by the applicable load factors, for all applicable load combinations.

The design shall be performed in accordance with the Eq. 551.5-1:

$$R_u \leq \phi R_n \quad (551.5-1)$$

where

R_u = Required strength

ϕ = Resistance factor specified in Section 552 through 557 and Appendix C-1

R_n = Nominal strength specified in Section 552 through 557 and Appendix C-1

ϕR_n = Design strength

551.5.1b Load Factors and Load Combinations for LRFD

Load factors and load combinations for LRFD shall be stipulated by Section C-3.3.1.1b of Section C-3.

551.7 Yield Stress and Strength Increase from Cold Work of Forming

551.7.1 Yield Stress

The yield stress used in design, F_y , shall not exceed the specified minimum yield stress of steels as listed in Section 551.2.2.1 or 551.2.3.2, as established in accordance with Section 556, or as increased for cold work of forming in Section 551.7.2.

551.7.2 Strength Increase from Cold Work of Forming.

Strength increase from cold work of forming shall be permitted by substituting F_{ya} for F_y , where F_{ya} is the average yield stress of the full section. Such increase shall be limited to Sections 553.2, 553.3.1 (excluding Section

553.3.1.1(b)), 553.3.4, 553.3.5, 554.4, and 554.6.1. The limits and methods for determining F_{ya} shall be in accordance with (a), (b) and (c).

1. For axially loaded compression members and flexural members whose proportions are such that the quantity p for strength determination is unity as determined in accordance with Section 552.2 for each of the component elements of the section, the design yield stresses, F_{ya} , of the steel shall be determined on the basis of one of the following methods:

- Full section tensile tests [see paragraph (a) of Section 556.3.1],
- Stub column tests [see paragraph (b) of Section 556.3.1],
- Computed in accordance with Eq. 551.7-1.

$$F_{ya} = CF_{yc} + (1 - C) F_{yf} \leq F_{uv} \quad (551.7-1)$$

where

F_{ya} = Average yield stress of full unreduced section of compression members or full flange sections of flexural members

C = For compression members, ratio of total corner cross-sectional area to total cross-sectional area of full section; for flexural members, ratio of total corner cross-sectional area of controlling flange to full cross-sectional area of controlling flange

$F_{yc} = B_c F_{yv} / (R/t)^m$, tensile yield stress of corners
(551.7-2)

Eq. 551.7-2 applies only when

$$F_{uv}/F_{yv} \geq 1.2, R/t \leq 7, \text{ and the included angle } \leq 120^\circ$$

where

$$B_c = 3.69 (F_{uv}/F_{yv}) - 0.819 (F_{uv}/F_{yv})^2 - 1.79 \quad (551.7-3)$$

F_{yv} = Tensile yield stress of virgin steel specified by Section 551.2 or established in accordance with Section 556.3.3

R = Inside bend radius

t = Thickness of section

$$m = 0.192 (F_{uv}/F_{yv}) - 0.068 \quad (551.7-4)$$

F_{uv} = Tensile strength of virgin steel specified by Section 551.2 or established in accordance with Section 556.3.3

F_{yf} = Weighted average tensile yield stress of flat portions established in accordance with Section 556.3.2 or virgin steel yield stress if tests are not made

2. For axially loaded tension members, the yield stress of the steel shall be determined by either method (1) or method (3) prescribed in paragraph (a) of this section.
3. The effect of any welding on mechanical properties of a member shall be determined on the basis of tests of full section specimens containing, within the gauge length, such welding as the manufacturer intends to use. Any necessary allowance for such effect shall be made in the structural use of the member.

551.8 Serviceability

A structure shall be designed to perform its required functions during its expected life. Serviceability limit states shall be chosen based on the intended function of the structure and shall be evaluated using realistic loads and load combinations.

551.9 Referenced Documents

The following documents or portions thereof are referenced in this Specification and shall be considered part of the requirements of this Specification.

1. American Iron and Steel Institute (AISI), 1140 Connecticut Ave., NW, Washington, DC 20036:

AISI S200-07, North American Standard for Cold-Formed Steel Framing – General Provisions

AISI S210-07, North American Standard for Cold-Formed – Floor and Roof System Design

AISI S211-07, North American Standard for Cold-Formed Steel Framing - Wall Stud Design

AISI S212-07, North American for Cold-Formed Steel Framing – Header Design

AISI S214-07, North American Standard for Cold-Formed Steel Framing- Truss Design

AISI S901-02*, Rotational Lateral Stiffness Test Method for Beam-to- Panel Assemblies

AISI S902-02, Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns

AISI S906-04, Standard Procedures for Panel and Anchor Structural Tests

*Note: * AISI test procedures previously designated as AISI TSn-xx are re-designated to AISI S9n-xx, where "n" is the test procedure sequence number and "xx" is the year the standard was developed or updated.*

2. American Society of Mechanical Engineers (ASME), 1828 L Street, NW, Washington, Dc 20036:

ASME B46.1-2000, Surface Texture, Surface Roughness, Waviness, and Lay

3. American Society for Testing and Materials (ASTM), 100 Barr Harbour Drive, West Conshohocken, Pennsylvania 19428-2959:

ASTM A36/ A36m-05, Standard Specification for Carbon Structural Steel

ASTM A194/A194M-06, Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service, or Both

ASTM A242/ A242 M-04e1, Standard Specification for High-Strength Low-Alloy Structural Steel

ASTM A307-04, Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength

ASTM A325-06, Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

ASTM A325M-05, Standard Specification for Structural Bolts, Steel, Heat Treated, 830 MPa Minimum Tensile Strength [Metric]

ASTM A354-04, Standard Specification For Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners

ASTM A370-05, Standard Specifications for Standard Test Methods and Definitions for Mechanical Testing of Steel Products

ASTM A449-04b, Standard Specification for Hex Cap Screws, Bolts, and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use.

ASTM A490-06, Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength

ASTM A490M-04a, Standard Specification for High Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]

ASTM A500-03a, Standard Specification for Cold Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M-05, Standard Specification for High Strength Carbon-Manganese Steel of Structural Quality

ASTM A563-04, Standard Specification for Carbon and Alloy Steel Nuts

ASTM A563M-04, Standard Specification for Carbon and Alloy Steel Nuts [Metric]

ASTM A572/A572M-06, Standard Specification for High-Strength Low Alloy Columbium-Vanadium Structural Steel

ASTM A588 / A588M-05, Standard Specification for High-Strength Low Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4-in. [100mm] Thick

ASTM A606-04, Standard Specification for Steel, Sheet and Strip, High-Strength, Low alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A653/ A653M-06, Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

ASTM A847 / A847M-05, Standard Specification for Cold Formed Welded and Seamless High Strength, Low alloy Structural Tubing with Improved Atmospheric Corrosion Resistance

ASTM A875 / A875M-05, Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process

ASTM A1003/ A1003M-05, Standard Specification for Steel Sheet, Carbon, Metallic and Non Metallic-Coated for Cold Formed Framing Members

ASTM A1008/ A1008M-05b, Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low Alloy, High-Strength Low Alloy with Improved Formability, Solution Hardened, and Bake Hardenable

ASTM A1011/A1011M-05a, Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low Alloy and High Strength Low alloy with Improved Formability

ASTM A1039/ A1039M-04, Standard Specification for Steel, Sheet, Hot-Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll-Casting Process

ASTM E1592-01, Standard Test Method for Structural Performance of Sheet Metal Roof and Siding System by Uniform Static Air Pressure Difference

ASTM F436-04, Standard Specification for Hardened Steel Washers

ASTM F436M-04, Standard Specification for Hardened Steel Washer [Metric]

ASTM F844-04, Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959-05a, Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners

ASTM F959M-04, Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners [Metric]

4. U.S. Army Corps of Engineers:CEGS-07416, Guide Specification for Military Construction, Structural Standing Seam Metal Roof (SSSMR) System, 1995
5. Factory Mutual, Corporate Offices, 1301 Atwood Avenue, P.O. Box 7500, Johnston, RI 02919: FM 4471, Approval Standard for Class 1 Metal Roofs, 1995

SECTION 552 ELEMENTS

552.1 Dimensional Limits and Considerations

552.1.1- Flange-Flat-Width-to-Thickness Considerations

1. Maximum Flat-Width-to-Thickness ratios, Maximum allowable overall flat-width-to-thickness ratios w/t , disregarding intermediate stiffeners and taking t as the actual thickness of the element, shall be determined in accordance with this section as follows:

- a. Stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by:

Simple lip, $w/t \leq 60$

Any other kind of stiffener

(i) when $I_s < I_a$, $w/t \leq 60$

(ii) when $I_s \geq I_a$, $w/t \leq 60$

where

I_s = Actual moment of inertia of full stiffener about its own centroidal axis parallel to element to be stiffened

I_a = adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element

- b. Stiffened compression element with both longitudinal edges connected to other stiffened elements, $w/t \leq 500$

- c. Unstiffened compression element, $w/t \leq 60$

It shall be noted that unstiffened compression elements that have w/t ratios exceeding approximately 30 and stiffened compression elements that have w/t ratios exceeding approximately 250 are likely to develop noticeable deformation at the full available strength, without affecting the ability of the member to develop the required strength.

Stiffened elements having w/t ratios greater than 500 provide adequate available strength to sustain the required loads; however, substantial deformations of such elements usually will invalidate the design equations of this Specification.

2. Flange Curling. Where the flange of a flexural member is usually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, Eq. 552.1-1 shall be permitted to be applied to compression and tension flange, either stiffened or unstiffened as follows:

$$w_f = \left(\sqrt{\frac{0.061tdE}{f_{av}}} \right)^4 \sqrt{\frac{100c_f}{d}} \quad (522.1-1)$$

where

w_f = width of flange projecting beyond web; or half of distance between webs for box- or U-type beams

t = flange thickness

d = depth of beam

f_{av} = average stress in full unreduced flange width. Where members are design by the effective design width procedure, the average stress equals the maximum stress multiplied by the ratio of the effective design width to the actual width)

c_f = amount of curling displacement

3. Shear Lag Effects – Short Spans Supporting Concentrated Loads. Where the beam has a span of less than $30w_f$ (w_f as defined below) and it carries one concentrated load, or several loads spaced farther apart than $2w_f$, the effective design width of any flange, whether in tension or compression, shall be limited by the values in Table 552-1.

Table 552-1 Short Span, Wide Flanges – Maximum Allowable Ratio of Effective Design Width (b) to Actual Width (w)

L/w_f	Ratio b/w	L/w_f	Ratio b/w
30	1.00	14	0.82
25	0.96	12	0.78
20	0.91	10	0.73
18	0.89	8	0.67
16	0.86	6	0.55

where

- L = full Span for simple beams; or the distance between inflection points for continuous beams;
 w_f = width of flange projection beyond web for I-beam and similar sections; or half the distance between webs for box-or U-type sections

For flanges of I-beams and similar sections stiffened by lips at the outer edges, w_f shall be taken as the sum of flange projection beyond the web plus the depth of the lip.

552.1.2 Maximum Web nDepth-To-Thickness Ratios

The ratio, h/t , of the webs of flexural members shall not exceed the following limits:

1. For unreinforced webs: $(h/t)_{max} = 200$
2. For webs which are provided with bearing stiffeners satisfying the requirements of Section 553.3.7a:
 - a. Where using bearing stiffeners only, $(h/t)_{max} = 260$
 - b. Where using bearing stiffeners and intermediate stiffeners, $(h/t)_{max} = 300$

where

- h = depth of flat portion of web measured along plane of web
 t = web thickness. Where a web consists of two or more sheets, the h/t ratio is computed for the individual sheets

552.2 Effective Widths of Stiffened Elements

552.2.1 Uniformly Compressed Stiffened Elements

1. Strength Determination

The effective width, b , shall be calculated from either Eq. 552.2-1 or Eq. 552.2-2 as follows:

$$b = w \quad \text{when } \lambda \leq 0.673 \quad (552.2-1)$$

$$b = \rho w \quad \text{when } \lambda > 0.673 \quad (552.2-2)$$

where

w = flat width as shown in Figure 522-1

ρ = Local reduction factor

$$= (1 - 0.22 / \lambda) / \lambda \quad (552.2-3)$$

λ = slenderness factor

$$= \sqrt{\frac{f}{F_{cr}}} \quad (552.2-4)$$

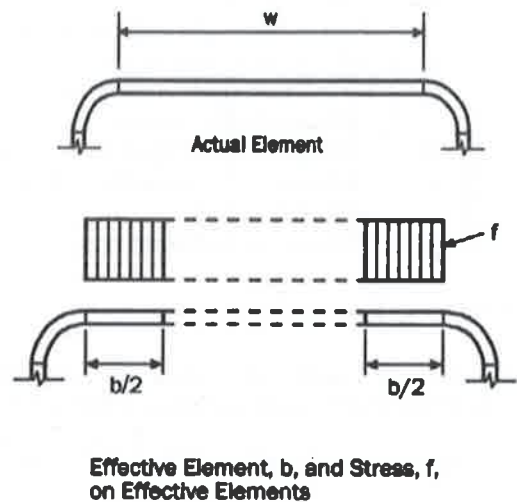


Figure 552-1 Stiffened Elements

where

f = stress in compression element computed as follows:

For flexural members:

- a. If Procedure I of Section 553.1.1 is used:

- When initial yielding is in compression in the element considered, $f = F_y$
- When the initial yielding is in tension, the compressive stress f in the element considered is determined on the basis of effective section at M_y (moment causing initial yield)

- b. If Procedure II of Section 553.1.1 is used, f is the stress in the elements considered at M_n determined on the basis of the effective section.

- c. If Section 553.1.2.1 is used, f is the stress F_c as described in that Section in determining effective section modulus S_c

For compression members, f is taken equal to F_n as determined in accordance with Section 533.4

$$F_{cr} = k \frac{\pi^2 E}{12(1 - \mu^2)} \left(\frac{t}{w} \right)^2 \quad (552.2-5)$$

where

- k = plate buckling coefficient
 = 4 for stiffened elements supported by a web on each longitudinal edge values for different types of elements are given in the applicable sections
 E = Modulus of Elasticity of steel
 t = thickness of uniformly compressed stiffened elements
 μ = Poisson's ratio of steel

2. Serviceability Determination

The effective width b_d used in determining serviceability shall be calculated from either as follows:

$$b_d = w \quad \text{when } \lambda \leq 0.673 \quad (552.2-6)$$

$$b_d = \rho w \quad \text{when } \lambda \leq 0.673 \quad (552.2-7)$$

where

- w = flat width
 ρ = Reduction factor determined by either of the following two procedures:

a. Procedure I:

A conservative estimate of the effective width is obtained from Eqs. 552.2-3 and 552.2-4 by substituting f_d for f , where f_d is the computed compressive stress in the element being considered.

b. Procedure II:

For stiffened elements supported by a web on each longitudinal edge, an improved estimate of the effective width is obtained by calculating ρ as follows:

$$\rho = 1 \quad \text{when } \lambda \leq 0.673$$

$$\rho = (1.358 - 0.461 / \lambda) / \lambda \quad \text{when}$$

$$0.673 < \lambda < \lambda_c$$

(552.2-8)

$$\rho = (0.41 + 0.59 \sqrt{F_y / f_d} - 0.22 / \lambda) / \lambda$$

$$\text{when } \lambda \geq \lambda_c$$

(552.2-9)

$$\rho \leq 1 \quad \text{for all cases}$$

where

λ = A value as defined by Eq. 552.2-4, except that f_d is substituted for f

$$\lambda = 0.256 + 0.328 (w / t) \sqrt{(F_y / E)} \quad (552.2-10)$$

552.2.2 Uniformly Compressed Stiffened with Circular or Non-Circular Holes

1. Strength Determination

For circular holes:

The effective width, b , shall be calculated by either Eq. 552.2-11 or Eq. 552.2-12 as follows:

$$\text{For } 0.50 \geq \frac{d_h}{w} \geq 0, \quad \text{and } \frac{w}{t} \leq 70,$$

distance between centers of holes $\geq 0.50w$ and $\geq 3d_h$

$$b = w - d_h \quad \text{when } \lambda \leq 0.673 \quad (552.2-11)$$

$$b = \frac{w \left(1 - \frac{0.22}{\lambda} - \frac{0.8d_h}{w} + \frac{0.085d_h}{w\lambda} \right)}{\lambda} \quad (552.2-12)$$

$$\text{when } \lambda \leq 0.673$$

In all cases, $b \leq w - d_h$

where

- w = flat width
 t = thickness of element
 d_h = diameter of holes
 λ = a value as defined in Section 552.2.1

For non-circular holes:

A uniformly compressed stiffened element with non-circular holes shall be assumed to consist of two unstiffened strips of flat width, c , adjacent to the holes (see Figure 552-2). The effective width, b , of each unstiffened strip adjacent to the hole shall be determined in accordance with 552.2.1 (a), except that plate buckling coefficient, k , shall be taken as 0.43 and was c . These provisions shall be applicable within the following limits:

- a. Center to center hole spacing, $s \geq 610 \text{ mm}$

- b. Clear distance from the hole at ends, $s_{end} \geq 254 \text{ mm}$,
 c. Depth of hole, $d_h \leq 63.5 \text{ mm}$,
 d. Length of hole, $L_h \leq 114 \text{ mm}$, and
 e. Ratio of the depth of hole, d_h , to the out-to-out width, w_o , $d_h/w_o \leq 0.5$.

Alternatively, the effective width, b , shall be permitted to be determined by stub-column tests in accordance with the test procedure, AISI S902.

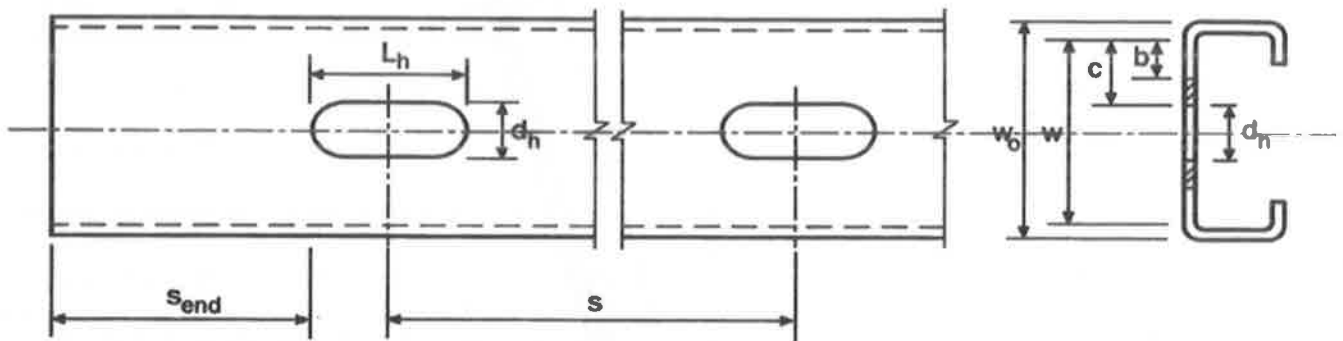


Figure 552-2 Uniformly Compressed Stiffened Elements with Non-Circular Holes

2. Serviceability Determination

The effective width, b_d , used in determining serviceability shall be equal to b calculated in accordance with Procedure I of Section 552.2.1(b), except that f_d is substituted for f , where f_d is the computed compressive stress in the element being considered.

552.2.3 Webs and Other Stiffened Elements Under Stress Gradient

The following notation shall apply in this section:

- b_1 = effective width, dimension defined in Figure 552-3
 b_2 = Effective width, dimension defined in Figure 552-3
 b_e = Effective width, b , determined in accordance with section 552.2.1.1, with f_1 substituted for f and with k determined as given in this section
 b_o = out-to-out width of the compression flange as defined in Figure 552-4
 f_1, f_2 = stresses shown in Figure 552-3 calculated on the basis of effective section. Where f_1 and f_2 are both compression, $f_1 \geq f_2$
 h_o = Out-to-out depth of web as defined in Figure 552-4
 k = plate buckling coefficient

$$\psi = |f_2/f_1| \text{ (absolute value)}$$

1. Strength Determination

- a. For webs under stress gradient (f_1 in compression and f_2 in tension as shown in Figure 552-3 (a), the effective widths and plate buckling coefficient shall be calculated as follows:

$$k = 4 + 2(1 + \psi)^3 + 2(1 + \psi) \quad (552.2-13)$$

For $h_o/b_o \leq 4$

$$b_1 = b_e/(3 + \psi) \quad (552.2-14)$$

$$b_2 = b_e/2 \quad \text{when } \psi > 0.236 \quad (552.2-15)$$

$$b_2 = b_e - b_1 \quad \text{when } \psi > 0.236 \quad (552.2-16)$$

In addition, $b_1 + b_2$ shall not exceed the compression portion of the web calculated on the basis of effective section.

For $h_o/b_o > 4$

$$b_1 = b_e / (3 + \psi) \quad (552.2-17)$$

$$b_2 = b_e / (1 + \psi) - b_1 \quad (552.2-18)$$

b. For other stiffened elements under stress gradient (f_1 and f_2 in compression as shown in Figure 522-3 (b))

$$k = 4 + 2(1 - \psi)3 + 2(1 - \psi) \quad (552.2-19)$$

$$b_1 = b_e / (3 - \psi) \quad (552.2-20)$$

$$b_2 = b_e - b_1 \quad (552.2-21)$$

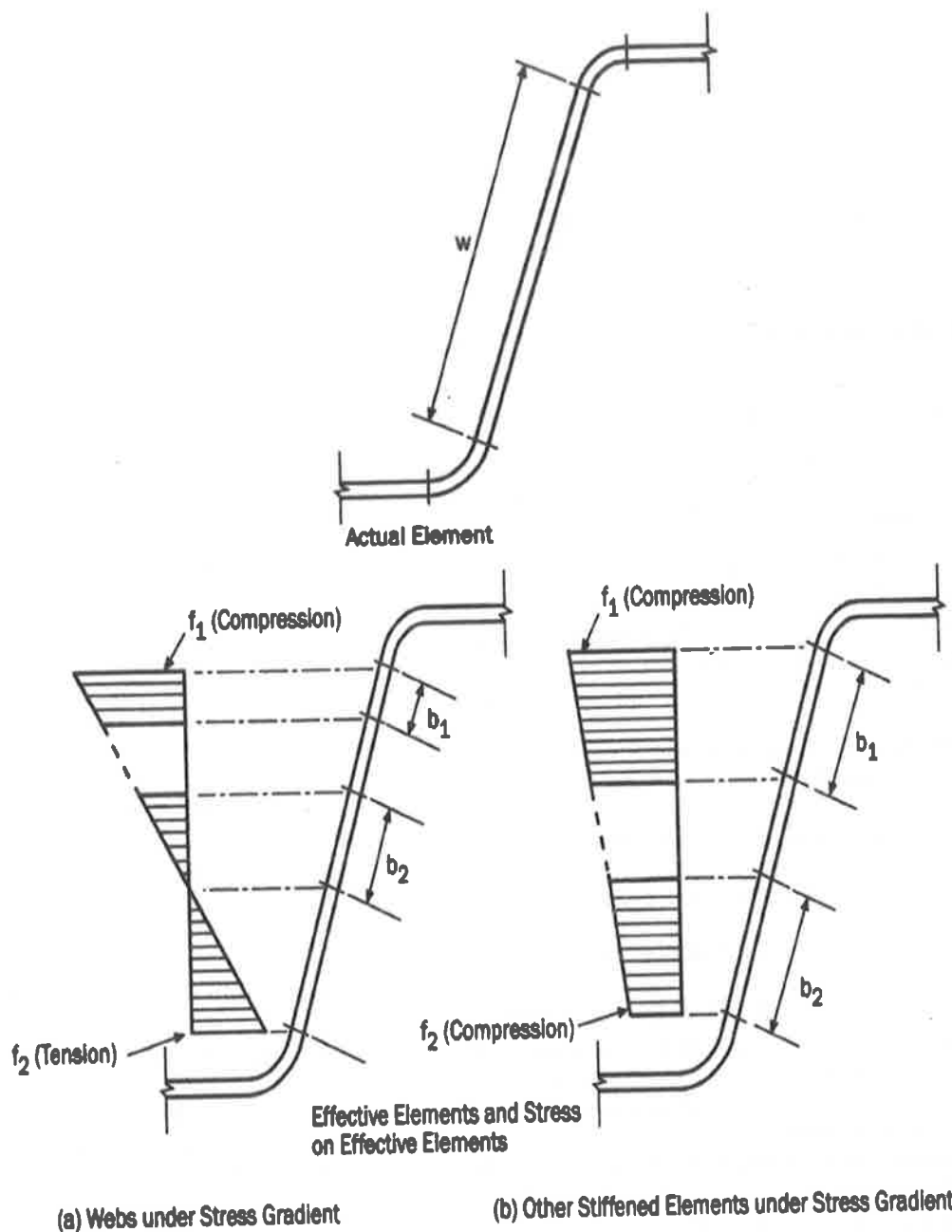


Figure 552-3 Webs and Other Stiffened Elements under Stress Gradient

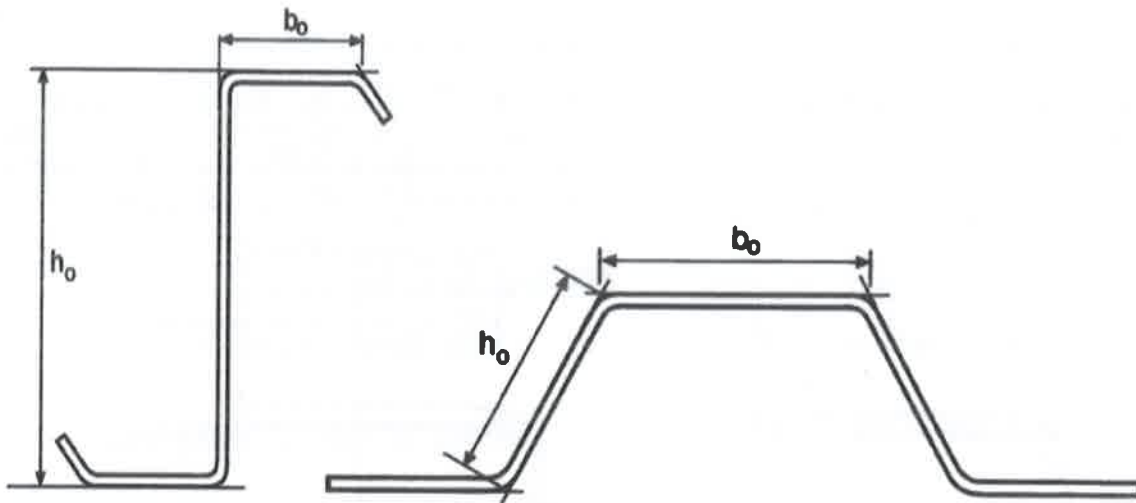


Figure 552-4 Out-to-Out Dimensions of Webs and Stiffened Elements under stress Gradient

1. Serviceability Determination

The effective widths used in determining serviceability shall be calculated in accordance with Section 552.3(a) except that f_{d1} and f_{d2} are substituted for f_1 and f_2 , where f_{d1} and f_{d2} are the computed stresses f_1 and f_2 based on the effective section at the load for which serviceability is determined.

552.2.4 C-Section Webs with Holes under Stress Gradient

The provisions of Section 552.2.4 shall apply within the following limits:

1. $d_h/h \leq 0.7$,
2. $H/t \leq 200$,
3. Holes centered at mid-depth of web,
4. Clear distance between holes $\geq 457\text{mm}$,
5. Non-circular holes, corner radii $\geq 2t$,
6. Non-circular holes, $d_h \leq 64\text{mm}$ and $L_h \leq 114\text{mm}$,
7. Circular holes, diameter $\leq 152\text{mm}$, and
8. $d_h \leq 14\text{mm}$.

where

- d_h = depth of web hole
 h = depth of flat portion of web measured along plane of web
 t = thickness of web
 L_h = length of web hole
 b_1, b_2 = effective widths defined by Figure 552-3
 Effective Widths of Unstiffened Elements
 Effective Widths of Unstiffened Elements

1. Strength Determination

- a. When $d_h/h < 0.38$, the effective widths b_1 and b_2 shall be determined in accordance with Section 552.2.3a by assuming no holes exist in the web.
- b. When $d_h/h < 0.38$, the effective width shall be determined in accordance with Section 552.3.1a, assuming the compression portion of the web consists of an unstiffened element adjacent to the hole with $f = f_1$ as shown in Figure 552-3.

2. Serviceability Determination

The effective widths shall be determined in accordance with Section 552.2.3b by assuming no hole exists in the web.

552.3 Effective Widths of Unstiffened Elements

552.3.1 Uniformly Compressed Unstiffened Elements

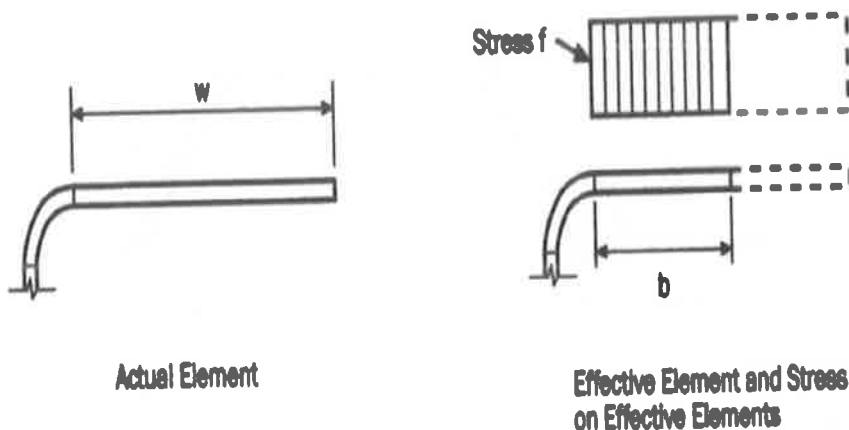


Figure 552-5 Unstiffened Element with Uniform Compression

1. Strength Determination

The effective width, b , shall be determined in accordance with Section 552.2.1a, except that plate buckling coefficient, k , shall be taken as 0.43 and w as defined in Figure 552-5.

2. Serviceability Determination

The effective width, b_d , used in determining serviceability shall be calculated in accordance with Procedure I of Section 552.2.1 (b), except that f_d is substituted for f and $k = 0.43$.

552.3.2 Unstiffened and Edge Stiffeners with Stress Gradient

The Following notation shall apply in this section:

- b = effective width measured from the supported edge, determined in accordance with Section 552.2.1a, with f equal to f_1 and with k and ρ being determined in accordance with this section
- b_o = overall width of unstiffened element of unstiffened C-section member as defined in Figure 552-8
- f_1, f_2 = stresses shown in Figures 552-6, 552-7, and 552-8 calculated on the basis of the gross section. Where f_1 and f_2 are both compression, $f_1 \geq f_2$

- h_o = overall depth of unstiffened C-section member as defined in Figure 552-8
- k = plate buckling coefficient defined in this section or, otherwise, as defined in Section 552.2.1a
- t = thickness of element
- w = flat width of unstiffened element, where $w/t \leq 60$
- ψ = $|f_1/f_2|$ (absolute value) (552.3-1)
- λ = slenderness factor defined in section 552.2.1(a) with $f = f_1$
- ρ = reduction factor defined in this section or, otherwise, as defined in Section 552.2.1(a)

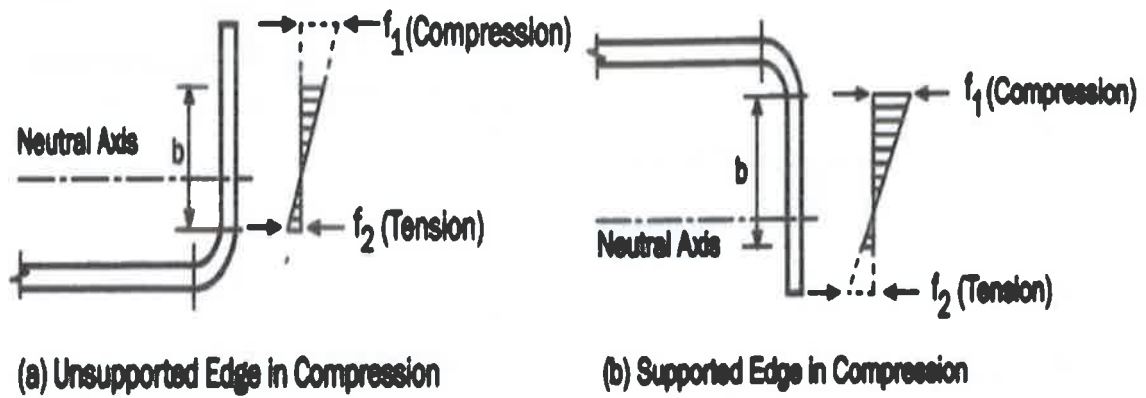


Figure 552-6 Unstiffened Element under Stress Gradient Both Longitudinal Edges in Compression

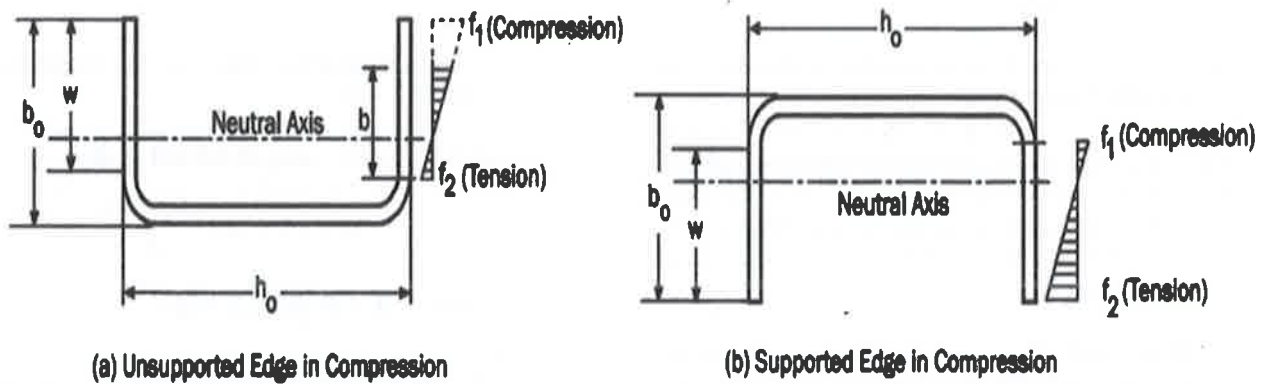


Figure 552-7 Unstiffened Elements under Stress Gradient One Longitudinal Edge in Compression and the Other Longitudinal Edge in Tension

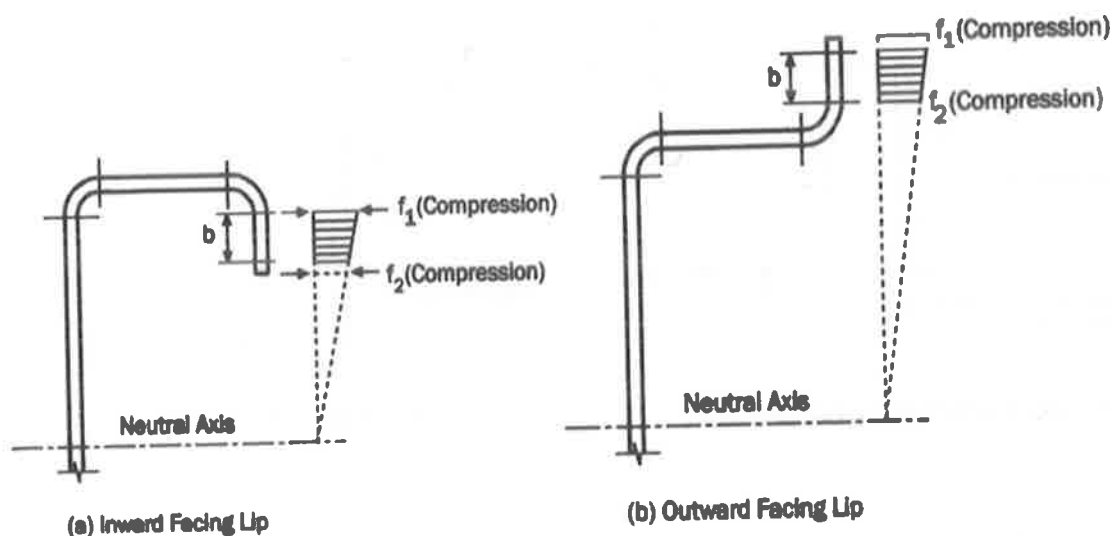


Figure 552-8 Unstiffened Elements of C Section under Stress Gradient for Alternative Methods

1. Strength Determination

The effective width, b , of an unstiffened element under stress gradient shall be determined in accordance with Section 552.2.1a with f equal to f_1 and the plate buckling coefficient, k , determined in accordance with this section, unless otherwise noted. For the cases where f_1 is in compression and f_2 is in tension, ρ in Section 552.2.1a shall be determined in accordance with this section.

- a. When both f_1 and f_2 are in compression (Figure 552-6), the plate buckling coefficient shall be calculated in accordance with either Eq. 552.3-2 or Eq. 552.3-3 as follows:

If the stress decreases toward the unsupported edge (Figure 552-6):

$$k = \frac{0.578}{\psi + 0.34} \quad (552.3-2)$$

If the stress increases toward the unsupported edge (Figure Be.2-1 (b)):

$$k = 0.57 - 0.21\psi + 0.07\psi^2 \quad (552.3-3)$$

- b. When f_1 is in compression and f_2 tension (Figure 552-7), the reduction factor and plate buckling coefficient shall be calculated as follows:

- c. If the unsupported edge is in compression Figure 552-7(a):

$$\rho = 1 \quad \text{when} \quad \lambda \leq 0.673(1 + \psi)$$

$$\rho = (1 + \psi) \frac{(1 + \psi)}{\lambda}$$

$$\text{when} \quad \lambda > 0.673(1 + \psi) \quad (552.3-4)$$

$$k = 0.57 + 0.21\psi + 0.07\psi^2 \quad (552.3-5)$$

- d. If the supported edge is in compression Figure 552-8(b):

For $\psi < 1$

$$\rho = 1 \quad \text{when} \quad \lambda \leq 0.673$$

$$\rho = (1 + \psi) \frac{\left(1 - \frac{0.22}{\lambda}\right)}{\lambda} + \psi$$

$$\text{when} \quad \lambda \leq 0.673 \quad (522.3-6)$$

$$k = 1.70 + 5\psi + 17.1\psi^2 \quad (522.3-7)$$

For $\psi < 1$, $\rho = 1$

The effective width, b , of the unstiffened elements of an unstiffened C-section member shall be permitted to be determined using the following alternative methods as applicable:

a. Alternative 1 for Unstiffened C-section:

When the unsupported edge is in compression and the supported edge is in tension (Figure 552-8 (a)):

$$b = w \quad \text{when} \quad \lambda \leq 0.856 \quad (552.3-8)$$

$$b = \rho w \quad \text{when} \quad \lambda \leq 0.856 \quad (552.3-9)$$

where

$$\rho = \frac{0.925}{\sqrt{\lambda}} \quad (552.3-10)$$

$$k = 0.145(b_o/h_o) + 1.256 \quad (552.3-11)$$

$$0.1 \leq b_o/h_o$$

b. Alternative 2 for Unstiffened C-sections:

When the supported edge is in compression and the unsupported edge in tension (Figure 552-8(b)), the effective width is determined in accordance with Section 552.2.3.

In calculating the effective section modulus S_e in section 553.3.1.1 or S_c in Section 553.3.1.1,

The extreme compression fiber in Figures 552-6 (b), and B3.2-3(a) shall be taken as the edge of the effective section closer to the unsupported edge. In calculating the effective section modulus S_e in Section 553.3.1, the extreme tension fiber in Figures 552-7(b) and 552-8(b) shall be taken as the edge of the effective section closer to the unsupported edge.

2. Serviceability Determination

The effective width b_d used in determining serviceability shall be calculated in accordance with Section 552.3.2(a), except that f_{d2} are substituted for f_1 and f_2 , respectively, where f_{d1} and f_{d2} are the computed stresses f_1 and f_2 as shown in Figures 552-6, 552-7, and 552-8, respectively, based on the gross section at the load for which serviceability is determined.

552.4 Effective Width of Uniformly Compressed Elements with a Simple Lip Edge Stiffener

The effective widths of uniformly compressed elements with a simple edge stiffener shall be calculated in accordance with (a) for strength determination and (b) for serviceability determination.

1. Strength Determination

For $w/t \leq 0.328S$:

$$I_a = 0 \quad (\text{no edge stiffener needed})$$

$$b = w \quad (552.4-1)$$

$$b_1 = b_2 = w/2 \quad (\text{see Figure 552-9}) \quad (552.4-2)$$

$$d_s = d'_s \quad (552.4-3)$$

For $w/t \leq 0.328S$

$$b_1 = (b/2)(RI) \quad (\text{see Figure 552-9}) \quad (552.4-4)$$

$$b_2 = (b - b_1) \quad (\text{see Figure 552-9}) \quad (552.4-5)$$

$$d_2 = d'_s(RI) \quad (552.4-6)$$

where

$$S = 1.28\sqrt{E/f} \quad (552.4-7)$$

w = flat dimension defined in Figure 552-9

t = thickness of section

I_a = adequate moment of inertia of stiffener, so that each component element will behave a stiffened element

$$= 399t^4 \left[\frac{(w/w)}{s} - 0.328 \right]^3 \leq t^4 \left[115 \frac{(w/t)}{s} \right] + 5 \quad (552.4-8)$$

b = effective design width

b_1, b_2 = Portions of effective design width as defined in Figure 552-9

d_s = Reduced effective width of stiffener as defined in Figure 552-9, and used in computing overall effective section properties

d'_s = Effective width of stiffener calculated in accordance with Section 552.3.2 (Figure 552-9)

$$(RI) = I_s/I_a \leq 1 \quad (552.4-9)$$

where

$$I_s = \text{Moment of inertia of full section of stiffener about parallel to element to be stiffened. For edge stiffeners, the round corner between stiffener and element to be stiffened is not considered as a part of the stiffener}$$

$$= (d_t^3 \sin^2 \theta) / 12 \quad (552.4-10)$$

See Figure 552-9 for Definitions of other Dimensional Variables.

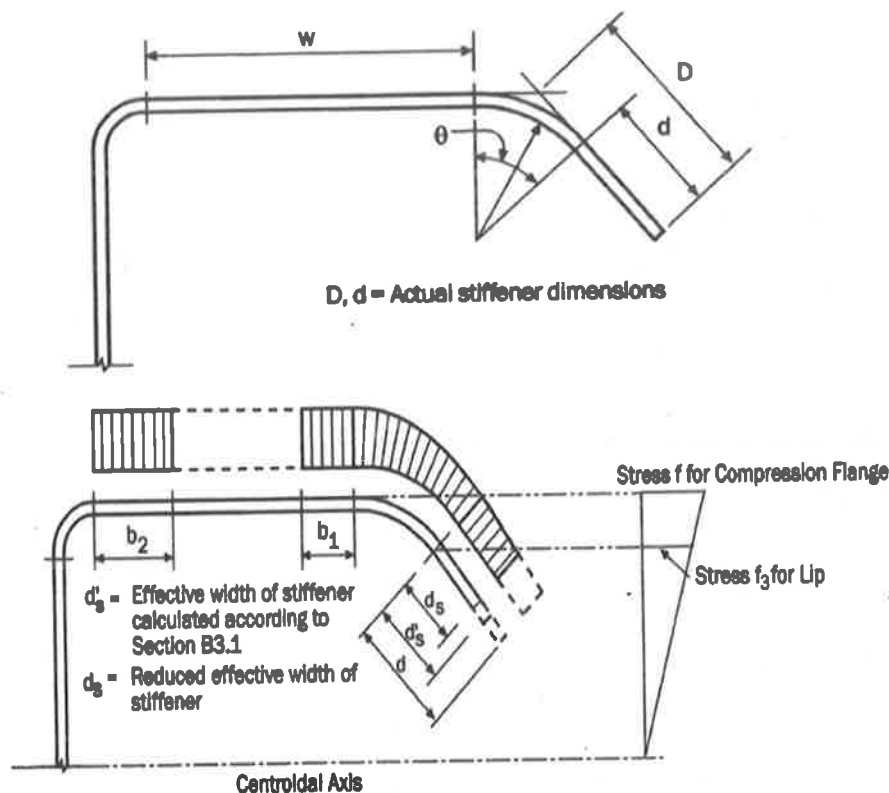


Figure 552-9 Elements with Simple Lip Edge Stiffener

The effective width, b , in Eq.552.4-4 and Eq.552.4-5 shall be calculated in accordance with Section 552.2.1 with the plate buckling coefficient, k , as given in Table 552-2 below:

Table 552-2 Determination of Plate Buckling Coefficient k

Simple Lip Edge Stiffener ($140^\circ \geq \theta \geq 40^\circ$)	
$D/w \leq 0.25$	$0.25 < D/w \leq 0.8$
$3.57(RI)^n + 0.43 \leq 4$	$(4.82 - (5D/w)(RI)^n) + 0.43 \leq 4$

where

$$n = \left(0.582 - \frac{(w/t)}{4S} \right) \geq \left(\frac{1}{3} \right) \quad (552.4-11)$$

2. Serviceability Determination

The effective width b_d , used in determining serviceability shall be calculated as in Section 552.4 except that f_d is substituted for f , where f_d is computed compressive stress in the effective section at the load for which serviceability is determined.

522.5 Effective Widths of Stiffened Elements with Single or Multiple Intermediate Stiffeners or Edge Stiffened Elements with Intermediate Stiffener(s)

522.5.1 Effective Widths of Uniformly Compressed Stiffened Elements with Single or Multiple Intermediate Stiffeners

The following notation shall apply as used in this section

- A_g = Gross area of element including stiffeners
- A_s = Gross area of stiffener
- b_e = Effective width of element, located at centroid of element including stiffeners; Figure 552-11
- b_o = Total flat width of stiffened element; see Figure 552-10
- b_p = Largest sub-element flat width; see Figure 552-10
- c_i = Horizontal distance from edge of element to centerline(s) of stiffener(s); see Figure 552-10
- F_{cr} = Plate elastic buckling stress
- f = Uniform compressive stress acting on flat element
- h = Width of elements adjoining stiffened element (e.g., depth of web in hat section with multiple intermediate stiffeners in compression flange is equal to h ; if adjoining elements have different widths, use smallest one)
- I_{sp} = Moment of inertia of stiffener about centerline of flat portion of element. The radii that connect the stiffener to the flat can be included
- k = Plate buckling coefficient of element
- k_d = Plate buckling coefficient for distortional buckling
- k_{loc} = Plate buckling coefficient for local sub-element buckling
- L_{br} = Unsupported length between brace points or Other restraints which restrict distortional Buckling of element
- R = Modification factor for distortional plate Buckling coefficient
- n = Number of stiffeners in element
- t = Element thickness
- i = Index for stiffener " i "
- λ = slenderness factor
- ρ = Reduction factor

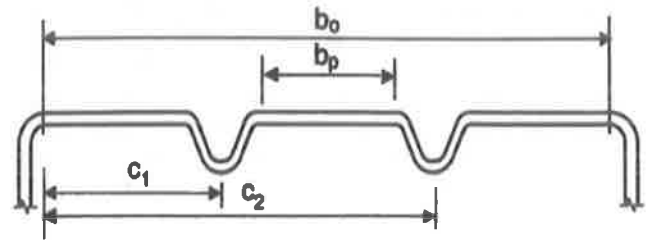


Figure 552-10 Plate Widths and Stiffener Locations



Figure 552-11 Effective Width Locations

The effective width shall be calculated in accordance with Eq. 552.5-1 as follows:

$$b_e = \rho \left(\frac{A_g}{t} \right) \quad (552.5-1)$$

where

$$\rho = 1 \quad \text{when } \lambda \leq 0.673$$

$$\rho = (1.0.22/\lambda)/\lambda \quad \text{when } \lambda > 0.673 \quad (552.5-2)$$

where

$$\lambda = \sqrt{\left(\frac{f}{F_{cr}} \right)} \quad (552.5-3)$$

where

$$F_{cr} = k \left(\frac{\pi^2 E}{12(1 - \mu^2)} \right) \left(\frac{t}{b_o} \right)^2 \quad (552.5-4)$$

The plate buckling coefficient, k , shall be determined from the minimum of R_{kd} and k_{loc} , as determined in accordance with Section 552.5.1.1 or 552.5.1.2, as applicable.

$$K = \text{the minimum of } R_{kd} \text{ and } k_{loc} \quad (552.5-5)$$

$$R = 2 \text{ when } b_o/h < 1$$

$$R = \left(\frac{11 - \frac{b_o}{h}}{5} \right) \geq \frac{1}{2} \text{ when } b_o/h \geq 1 \quad (552.5-6)$$

522.5.1.1 Specific Cases: Single or Identical Stiffeners, Equally Spaced

For uniformly compressed elements with single, or multiple identical and equally spaced stiffeners, the plate buckling coefficients and effective widths shall be calculated as follows:

1. Strength Determination

$$k_{loc} = 4(n + 1)^2 \quad (552.5-7)$$

$$k_d = \left(\frac{(1 + \beta^2)^2 + \gamma(1 + n)}{\beta^2(1 + \delta(n + 1))} \right) \quad (552.5-8)$$

where

$$\beta = (1 + \gamma(n + 1))^{1/4} \quad (552.5-9)$$

where

$$\gamma = \left(\frac{10.92 I_{sp}}{b_o t^3} \right) \quad (552.5-10)$$

$$\delta = \left(\frac{A_s}{b_o t} \right) \quad (552.5-11)$$

If $L_{br} < \beta b_o$, L_{br}/b_o shall be permitted to be substituted for β to account for increased capacity due to bracing.

2. Serviceability Determination

The effective width, b_d , used in determining serviceability shall be calculated as in Section 552.5.1.1, except that f_d is substituted for f , where f_d is the computed compressive stress in the element being considered based on the effective section at the load for which serviceability is determined.

552.5.1.2 General Cases: Arbitrary Stiffener Size, Location, and Number

For uniformly compressed stiffened elements with stiffeners of arbitrary size, location and number, the plate buckling coefficients and effective widths shall be calculated as follows:

1. Strength Determination

$$k_{loc} = 4 \left(\frac{b_o}{b_p} \right)^2 \quad (552.5-12)$$

$$k_d = \frac{((1 + \beta^2)^2 + 2 \sum_{i=1}^n \gamma_i \omega_i)}{(\beta^2(1 + 2 \sum_{i=1}^n \delta_i \omega_i))} \quad (552.5-13)$$

where

$$\beta = \left(2 \sum_{i=1}^n \gamma_i \omega_i + 1 \right)^{1/4} \quad (552.5-14)$$

where

$$\gamma_i = \left(\frac{10.92(I_{sp})_i}{b_o t^3} \right) \quad (552.5-15)$$

$$\omega_i = \sin^2 \left(\pi \frac{c_i}{b_o} \right) \quad (552.5-16)$$

$$\delta_i = \frac{(A_s)_i}{b_o t} \quad (552.5-17)$$

If $L_{br} < \beta b_o$, L_{br}/b_o shall be permitted to be substituted for β to account for increased capacity due to bracing.

2. Serviceability Determination

The effective width b_d used in determining serviceability shall be calculated as in Section 552.5.1.2.1, except that f_d is substituted for f , where f_d is the computed compressive stress in the element being considered based on the effective section at the load for which serviceability is determined.

552.5.2 Edge Stiffened Elements with Intermediate Stiffener(s)

1. Strength Determination

For edge stiffened elements with intermediate stiffener(s), the effective width, b_o shall be determined as follows:

- a. If $b_o/t \leq 0.328S$, the element is fully effective and no local buckling required.
- b. If $b_o/t > 0.328S$, then the plate buckling coefficient, k , is determined in accordance with Section 552.4, but with b_o replacing w in all expressions:

If k calculated from Section 552.4 is less than $4.0 (k < 4)$, the intermediate stiffener(s) is ignored and the provisions of Section 522.4 are followed for calculation of the effective width.

If k calculated from Section 552.4 is equal to $4.0 (k = 4)$, the effective width of the edge stiffened element is calculated from the provisions of Section 552.5.1, with the following exception:

R calculated in accordance with Section 552.5.1 is less than or equal to 1.

where

b_o = total flat width of edge stiffened element
see Sections 552.4 and 522.5.1 for
definitions of other variables

2. Serviceability Determination

The effective width, b_d , used in determining serviceability shall be calculated as in Section 552.5(b), except that f_d is substituted for f , where f_d is the computed compressive stress in the element being considered based on the effective section at the load for which serviceability is determined.

SECTION 553 MEMBERS

553.1 Properties of Sections

Properties of sections (cross-sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design. Properties shall be based on the full cross-section of the members (or net sections where the use of net section is applicable) except where the use of a reduced cross-section, or effective design width, is required.

553.2 Tension Members

See Section 553.3.5 of Section 553.3.

553.3 Flexural Members

553.3.1 Bending

The nominal flexural strength, M_n , shall be the smallest of the values calculated in accordance with sections 553.3.1.1, 553.3.1.2, 553.3.1.3, 553.3.1.4, 554.6.1.1, 554.6.1.2, and 554.6.2.1, where applicable.

See Section 553.3.6, as applicable for laterally unrestrained flexural members subjected to both bending and torsional loading, such as loads that do not pass through the shear center of the cross-section, a condition which is not considered in the provision of this section.

553.3.1.1 Nominal Section Strength

The nominal flexural strength, M_n , shall be calculated either on the basis of initiation of yielding of the effective section (Procedure I) or on the basis of the inelastic reserve capacity (Procedure II), as applicable. The applicable safety factors and the resistance factors given in this section shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

For sections with stiffened or partially compression Flange:

$$\Omega_b = 1.67 \text{ (ASD)} \qquad \phi_b = 0.95 \text{ (LRFD)}$$

For sections with unstiffened compression flanges:

$$\Omega_b = 1.67 \text{ (ASD)} \qquad \phi_b = 0.90 \text{ (LRFD)}$$

1. Procedure I –Based on Initiation of Yielding

The nominal flexural strength, M_n , for the effective yield moment shall be calculated in accordance with Eq. 553.3-1 as follows:

$$M_n = S_e F_y \quad (553.3-1)$$

where

S_e = Elastic section modulus of effective section calculated relative to extreme compression or tension fiber at F_y

F_y = Design yield stress determined in accordance with Section 551.7.1

2. Procedure II – Based on Inelastic Reserve Capacity

The inelastic flexural reserve capacity shall be permitted to be used when the following conditions are met:

- The member is not subject to twisting or to lateral, torsional, or flexural-torsional buckling.
- The effect of cold work of forming is not included in determining the yield stress F_y .
- The ratio of the depth of the compressed portion of the web to its thickness does not exceed λ_1 .
- The shear force does not exceed $0.35F_y$ for ASD, and $0.6F_y$ for LRFD times the web area (h_t for stiffened elements or w_t for unstiffened elements).
- The angle between any web and the vertical does not exceed 30.

The nominal flexural strength, M_n , shall not exceed either $1.25 S_e F_y$, as determined in accordance with Procedure I of Section 553.3.3(a) or that causing a maximum compression strain of C_{yey} (no limit is placed on the maximum tensile strain).

where

h = flat depth of web

t = base steel thickness of element

e_y = yield strain

$$= F_y / E$$

w = elements flat width

E = modulus of elasticity

C_y = compression strain factor calculated as follows:

- Stiffened compression elements without intermediate stiffeners

For compression elements without intermediate stiffeners, C_y , shall be calculated as follows:

$$C_y = 3 \text{ when } w/t \leq \lambda_1$$

$$C_y = 3 - 2 \left(\frac{\left(\frac{w}{t} \right) - \lambda_1}{\lambda_2 - \lambda_1} \right) \text{ when } \lambda_1 < w/t < \lambda_2 \quad (553.3-2)$$

where

$$\lambda_1 = \frac{1.11}{\sqrt{F_y/E}} \quad (553.3-3)$$

where

$$\lambda_2 = \frac{1.28}{\sqrt{F_y/E}} \quad (553.3-4)$$

- Unstiffened compression elements

For unstiffened compression elements, C_y , shall be calculated as follows:

- Unstiffened compression elements under stress gradient causing compression at one longitudinal edge and tension at the other longitudinal edge:

$$C_y = 3.0 \quad \text{when } \lambda \leq \lambda_3$$

$$C_y = 3 - 2[(\lambda - \lambda_3)/(\lambda_4 - \lambda_3)] \quad (553.3-5)$$

$$\text{when } \lambda_3 < \lambda < \lambda_4$$

$$C_y = 1 \quad \text{when } \lambda \geq \lambda_4$$

where

$$\lambda_3 = 0.43$$

$$\lambda_4 = 0.673(1 + \psi) \quad (553.3-6)$$

ψ = a value defined in Section 552.3.2

- Unstiffened compression elements under stress gradient causing compression at both longitudinal edges:

$$C_y = 1$$

- (ii-3) Unstiffened compression elements under uniform compression:

$$C_y = 1$$

- (iii) Multiple-stiffened compression elements and compression elements with edge stiffeners For multiple-stiffened compression elements and compression elements with edge stiffeners, C_y , shall be taken as follows:

$$C_y = 1$$

When applicable, effective design widths shall be used in calculating section properties. M_n shall be calculated considering equilibrium of stresses, assuming an ideally elastic-plastic stress-strain curve, which is the same in tension as in compression, assuming small deformation, and assuming that plane sections remain plane during bending. Combined bending and web crippling shall be checked by the provisions of Section 553.3.3.5.

553.3.1.2 Lateral-Torsional Buckling Strength

The provisions of this section shall apply to members with either an open cross-section as specified in Section 553.3.1(b.1) or closed box sections as specified in Section 553.3.1(b.2).

Unless otherwise indicated, the following safety factor and resistance factors and the nominal strengths calculated in accordance with Sections 553.3.1(b.1) and 553.3.1(b.2) shall be used to determine the allowable flexural strength or design flexural strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$\Omega_b = 1.67 \text{ (ASD)} \quad \phi_b = 0.90 \text{ (LRFD)}$$

553.3.3.1.2.1 Lateral- Torsional Buckling Strength of Open Cross-Section Members

The provisions of this section shall apply to I-, Z-, C-, and other singly-symmetric section flexural members (not including multiple-web deck, U- and closed box-type members, and curved or arch members) subject to lateral-torsional buckling. The provisions of this section shall not apply to laterally unbraced compression flanges of otherwise laterally stable sections. See Section 554.6.1a for C- and Z-purlins in which the tension flange is attached to sheathing.

For laterally unbraced segments of singly-, doubly-, and point-symmetric sections subject to lateral-torsional buckling, the nominal flexural strength, M_n , shall be calculated in accordance with Eq. 553.3-7)

$$M_n = S_c F_c \quad (553.3-7)$$

where

S_c = Elastic section modulus of effective section calculated relative to extreme compression fiber at F_c

1. F_c shall be determined as follows:

$$\text{For } F_e \geq 2.78 F_y$$

The member segment is not subject to lateral-torsional buckling at bending moments less than or equal to M_y . The available flexural strength shall be determined in accordance with Section 553.3.1a.

2. For $2.78 F_y > F_e > 0.56 F_y$

$$F_c = \frac{10}{9} F_y \left(1 - \frac{10 F_y}{36 F_e} \right) \quad (553.3-8)$$

3. For $F_e \leq 0.56 F_y$

$$F_c = F_e \quad (553.3-9)$$

where

F_y = design yield stress as determined in accordance with Section 551.7.1

F_e = elastic critical lateral-torsional buckling stress calculated in accordance with (a) or (b)

- a. For singly-, doubly-, and point-symmetric sections:

- (i) For bending about the symmetry axis:

$$F_e = \frac{C_b r_o A}{S_f} \sqrt{\sigma_{ey} \sigma_t} \quad (553.3-10)$$

for singly- and doubly-symmetric sections

$$F_e = \frac{C_b r_o A}{2 S_f} \sqrt{\sigma_{ey} \sigma_t}$$

For point-symmetric sections (553.3-11)

where

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad (553.3-12)$$

where

- M_{max} = Absolute value of maximum moment in unbraced segment
 M_A = Absolute value of moment at quarter point of unbraced segment
 M_B = Absolute value of moment at centerline of unbraced segment
 M_C = Absolute value of moment at three-quarter point of unbraced segment
 C_b = shall be permitted to be conservatively taken as unity for all cases. For cantilevers or overhangs where the free end is unbraced, C_b shall be taken as unity.
 r_o = Polar radius of gyration of cross-section about shear center

$$= \sqrt{r_x^2 + r_y^2 + x_o^2} \quad (553.3-13)$$

where

- r_x, r_y = Radii of gyration of cross-section about centroid principal axes
 x_o = Distance from shear center to centroid along principal x-axis, taken as negative
 A = Full unreduced cross-sectional area
 S_f = Elastic section modulus of full unreduced section relatively to extreme compression fiber

$$\sigma_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L_y}{r_y}\right)^2} \quad (553.3-14)$$

where

- E = modulus of elasticity of steel
 K_y = effective length factors for bending about y-axis
 L_y = unbraced length of member for bending about y-axis

$$\sigma_t = \frac{1}{Ar_o^2} \left(GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2} \right) \quad (553.3-15)$$

where

- G = Shear modulus
 J = Saint-Venant torsion constant of cross-section
 C_w = Torsional warping constant of cross-section
 K_t = Effective length factors for twisting
 L_t = Unbraced length of member for twisting

For singly-symmetric sections, x-axis of symmetry oriented such that the shear center has a negative x-coordinate.

For point-symmetric sections, such as Z-sections, x-axis shall be the centroidal axis perpendicular to the web.

Alternatively, F_e shall be permitted to be calculated using the equation given in (b) for doubly-symmetric I-sections, singly-symmetric C-sections, or point-symmetric Z-sections.

- (ii) For singly-symmetric sections bending about the centroidal axis perpendicular to the axis of symmetry:

$$F_e = \frac{C_s A \sigma_{ex}}{C_{TF} S_f} \left(j + C_s \sqrt{j^2 + r_o^2 \left(\frac{\sigma_t}{\sigma_{ex}} \right)} \right) \quad (553.3-16)$$

where

- C_s = +1 for moment causing compression on shear center of side of centroid
 = -1 for moment causing tension on shear center side of centroid

$$\sigma_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L_x}{r_x}\right)^2} \quad (553.3-17)$$

where

- K_x = Effective length factors for bending about x-axis
 L_x = Unbraced length of member for bending about x-axis
 $C_{TF} = 0.6 - 0.4(M_1/M_2)$ (553.3-18)

where

M_1, M_2 = the smaller and the larger bending moment, respectively, at the ends of the unbraced length in the plane of bending; M_1 and M_2 , the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at the both ends of this length, C_{TF} shall be taken as unity.

$$j = \frac{1}{2I_y} \left[\int_A x^3 dA + \int_A xy^2 dA \right] - x_o \quad (553.3-19)$$

- b. For I- sections. Singly-symmetric C-sections, or Z- sections bent about the centroidal axis perpendicular to the web (axis), the following equations shall be permitted to be used in lieu of (a) to calculate F_e :

$$F_e = \frac{C_b \pi^2 E d I_{yc}}{S_f (K_y L_y)^2} \quad (553.3-20)$$

For doubly-symmetric I-sections and singly-symmetric C-sections

$$F_e = \frac{C_b \pi^2 E d I_{yc}}{2 S_f (K_y L_y)^2} \quad (553.3-21)$$

For point symmetric Z- sections

where

d = Depth of section
 I_{yc} = Moment of inertia of compression portion of section about centroidal axis of entire section parallel to web, using full unreduced section see (a) for definition of other variables

553.3.1.2.2 Lateral-Torsional Buckling Strength of Closed Box Members

For closed box members, the nominal flexural strength, M_n , shall be determined in accordance with this section.

If the laterally unbraced length of the member is less than or equal to L_u , the nominal flexural strength shall be determined in accordance with Section 553.3.1.1. L_u shall be calculated as follows:

$$L_u = \frac{0.36 C_b \pi}{F_y S_y} \sqrt{E G J I_y} \quad (553.3-22)$$

See Section 553.3.1.2.1 for definition of variables.

If the laterally unbraced length of a member is larger than L_u , as calculated in Eq. 553.1-22, the nominal flexural strength shall be determined in accordance with Section 553.3.1.2.1, where the critical lateral-torsional buckling stress, F_e , is calculated as follows:

$$F_e = \frac{C_b \pi}{K_y L_y S_f} \sqrt{E G J I_y} \quad (553.3-23)$$

where

J = torsional constant of box section
 I_y = moment of inertia of full unreduced section about centroidal axis parallel to web

See Section 553.3.1b.1 for definition of other variables.

553.3.1.3 Flexural Strength of Closed Cylindrical Tubular Members

For closed cylindrical tubular members having a ratio of outside diameter to wall thickness, D/t , not greater than $0.441 E/F_y$, the nominal flexural strength [moment resistance], M_n , shall be calculated in accordance with Eq. 553.3-24. the safety factor and resistance factors given in this section shall be used to determine the allowable flexural strength or design flexural strength [factored moment resistance] in accordance with the applicable design method in Section 551.4, or Section 551.5.

$$M_n = F_c S_f \quad (553.3-24)$$

$$\phi_b = 0.95 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

For $D/t \leq 0.0714 E/F_y$

$$F_c = 1.25 F_y \quad (553.3-25)$$

For $0.0714 E/F_y < D/t \leq 0.318 E/F_y$

$$F_c \left[0.970 + 0.020 \left(\frac{E/F_y}{D/t} \right) \right] F_y \quad (553.3-26)$$

For $0.318 E/F_y < D/t \leq 0.441 E/F_y$

$$F_c = 0.328 E/(D/t) \quad (553.3-27)$$

where

 D = outside diameter of cylindrical tube t = thickness F_c = critical flexural buckling stress S_f = elastic section modulus of full unreduced cross section relative to extreme compression fiber

See Section 553.3.1b.1 for definitions of other variables.

553.3.1.4 Distortional Buckling Strength

The provisions of this section shall apply to I-, Z-, C-, and other open cross-section members that employ compression flanges with edge stiffeners, with the exception of members that meet the criteria of Section 554.6.1.1, 554.6.1.2 when the R factor of Eq. 554.6.1.2-1 is employed, or 554.6.2.1. The nominal flexural strength shall be calculated in accordance with Eq. 553.1-28 or Eq. 553.3-29. The safety factor and resistance factors given in this section shall be used to determine the allowable flexural strength or design flexural strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

For $\lambda_d \leq 0.673$

$$M_n = M_y \quad (553.3-28)$$

For $\lambda_d > 0.673$

$$M_n = \left(1 - 0.22 \left(\frac{M_{crd}}{M_y} \right)^{0.5} \right) \left(\frac{M_{crd}}{M_y} \right)^{0.5} M_y \quad (553.3-29)$$

where

$$\lambda_d = \sqrt{\frac{M_y}{M_{crd}}} \quad (553.3-30)$$

$$M_y = S_{fy} F_d \quad (553.3-31)$$

where

 S_{fy} = Elastic section modulus of full unreduced section relative to extreme compression fiber F_d = Elastic distortional buckling stress calculated in accordance with either Section 553.3.1d (a), (b), or (c)

1. Simplified Provisions for Unrestrained C- and Z-Sections with Simple Lip Stiffeners

For C- and Z- sections that have no rotational restraint of the compression flange and are within the dimensional limits provided in this section, Eq. 553.3-32 shall be permitted to be used to calculate a conservative prediction of the distortional buckling stress, F_d . See section 553.3.1d(b) or 553.3.1d(c) for alternative provisions and for members outside the dimensional limits of this section.

The following dimensional limits shall apply:

- a. $50 \leq h_o/t \leq 200$,
- b. $25 \leq b_o/t \leq 100$,
- c. $6.25 < D/t \leq 50$,
- d. $45^\circ \leq \theta < 90^\circ$,
- e. $2 \leq h_o/b_o \leq 8$, and
- f. $0.04 \leq D \sin \theta / b_o \leq 0.5$

where

 h_o = Out-to-out web depth as defined in Figure 552-4 t = Base steel thickness b_o = Out-to-out flange width as defined in Figure 552-4 D = Out-to-out lip dimension as defined in Figure 552-9 θ = Lip angle as defined in Figure 552-9

The distortional buckling stress, F_d , shall be calculated as follows:

$$F_d = \beta k_d \frac{\pi^2 E}{12(1 - \mu^2)} \left(\frac{t}{b_o} \right)^2 \quad (553.3-32)$$

where

$$\begin{aligned}\beta &= \text{A value accounting for moment gradient, which is permitted to be a conservatively taken as 1.0} \\ &= 1.0 \leq 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \leq 1.3\end{aligned}\quad (553.3-33)$$

where

L = Minimum of L_{cr} and L_m

where

$$L_{cr} = 1.2h_o \left(\frac{b_o D \sin \theta}{h_o t} \right)^{0.6} \leq 10h_o \quad (553.3-34)$$

where

L_m = Distance between discrete restraints that restrict distortional buckling (for continuously restrained members $L_{cr} = L_m$)

M_1, M_2 = The smaller and the larger end moment, respectively, in the unbraced segment (L_m) of the beam; M_1/M_2 is negative when the moments cause reverse curvature and positive when bent in single curvature

$$k_d = 0.5 \leq 0.6 \left(\frac{b_o D \sin \theta}{h_o t} \right)^{0.7} \leq 8.0 \quad (553.3-35)$$

where

E = Modulus of Elasticity

μ = Poisson's ratio

2. For C- and Z- Sections or any Open Section with a Stiffened Compression Flange Extending to One Side of the Web where the Stiffener is either a Simple Lip or a Complex Edge Stiffener

The provisions of this section shall be permitted to apply to any open section with a single web and single edge stiffened compression flange; including those meeting the geometric limits of Section 553.3.1d (a). The distortional buckling stress, F_d , shall be calculated in accordance with Eq. 553.3-36) as follows:

$$F_d = \beta \frac{k_{\phi fe} + F_{\phi we} + k_{\phi}}{\bar{k}_{\phi fg} + \bar{k}_{\phi wg}} \leq 8.0 \quad (553.3-36)$$

where

β = A value accounting for moment gradient, which is permitted to be a conservatively taken as 1.0

$$= 1.0 \leq 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \leq 1.3 \quad (553.3-37)$$

where

L = Minimum of L_{cr} and L_m

where

$$\begin{aligned}L_{cr} = & \left(\left(\frac{4\pi^2 h_o (1 - \omega^2)}{t^3} \right) \left(I_{xf}(x_o - h_x)^2 \right. \right. \\ & \left. \left. + C_{wf} - \frac{I_{xyf}^2}{I_{yfy}^{1/4}} (x_o - h_x)^2 \right) \right. \\ & \left. + \frac{\pi^4 h_o^4}{720} \right)^{1/4}\end{aligned}\quad (553.3-38)$$

where

h_o = Out-to-out web depth as defined in Figure 552-4

μ = Poisson's ratio

t = Base steel thickness

I_{xf} = x-axis moment of inertia of the flange

x_o = x distance from the centroid of the flange to the shear center of the flange

h_x = x distance from the centroid of the flange to the flange/web junction

C_{wf} = Warping torsion constant of the flange

I_{xyf} = Product of the moment of inertia of the flange

I_{yf} = y-axis moment of inertia of the flange

In the above, I_{xf} , I_{yf} , I_{xyf} , C_{wf} , x_o , and h_x are properties of the compression flange plus stiffener about an x-y axis system located at the centroid of the flange, with the x-axis measured positive to the right from the centroid, and the y-axis positive down from the centroid.

L_m = Distance between discrete restraints that restrict distortional buckling (for continuously restrained members

$$L_{cr} = L_m)$$

M_1, M_2 = The smaller and the larger end moment, respectively, in the unbraced segment (L_m) of the beam; M_1/M_2 is negative when the moments cause reverse curvature and positive when bent in single curvature

$k_{\phi fe}$ = Elastic rotational stiffness provided by the flange to the flange / web juncture

$$\begin{aligned} & (\pi/L)^4 \left(EI_{xf}(x_o - h_x)^2 + EC_{wf} - \right. \\ & \left. = E \frac{I_{xyf}^2}{I_{yf}} (x_o - h_x)^2 \right) + (\pi/L)^2 GJ_f \end{aligned} \quad (553.3-3)$$

where

E = Modulus of elasticity of steel

G = Shear modulus

J_f = St. Venant torsion constant of the compression flange, plus edge Stiffener about an x-y axis located at the centroid of the flange, with the x-axis measured positive to the right from the centroid, and the y-axis positive down from the centroid

$k_{\phi we}$ = Elastic rotational stiffness provided by the web to the flange / web juncture

$$\begin{aligned} & = \frac{Et^3}{12(1-\mu^2)} \left(\frac{3}{h_o} + \left(\frac{\pi}{L} \right)^2 \frac{19h_o^2}{60} \right. \\ & \quad \left. + \left(\frac{\pi}{L} \right)^4 \frac{h_o^3}{240} \right) \end{aligned} \quad (553.3-40)$$

k_{ϕ} = Rotational stiffness provided by a restraining element (brace, panel, sheathing) to the flange / web juncture of a member (zero if the compression flange is unrestrained)

$\bar{k}_{\phi fg}$ = Geometric rotational stiffness (divided by the stress F_d) demanded by the flange from the flange / web juncture

$$\begin{aligned} & = \frac{\pi^2}{L} \left(A_f \left((x_o - h_x)^2 \left(\frac{I_{xyf}}{I_{yf}} \right)^2 - \right. \right. \\ & \quad \left. \left. 2y_o(x_o - h_x) \left(\frac{I_{xyf}}{I_{yf}} \right) + h_x^2 + y_o^2 \right) + \right. \\ & \quad \left. I_{xf} + I_{yf} \right) \end{aligned} \quad (553.3-41)$$

where

A_f = Cross-sectional area of the compression flange plus edge stiffener about an x-y axis located at the centroid of the flange, with the x-axis measured positive to the right from the centroid, and the y-axis positive down from the centroid

y_o = y distance from the centroid of the flange to the shear center of the flange

$\bar{k}_{\phi wg}$ = Geometric rotational stiffness (divided by the stress F_d) demanded by the flange from the flange / web juncture

$$k_{11} = \left[45360(1 - \xi_{web}) + 62160 \left(\frac{L}{h_o} \right)^2 \right]$$

$$k_{12} = 448\pi^2$$

$$k_{13} = \left(\frac{h_o}{L} \right)^2 [53 + 3(1 - \xi_{web})] \pi^4$$

$$\begin{aligned} & = \frac{h_o t \pi^2}{13440} \left(\frac{k_{11} + k_{12} + k_{13}}{\pi^4 + 28\pi^2 \left(\frac{L}{h_o} \right)^2 + 420 \left(\frac{L}{h_o} \right)^4} \right) \end{aligned} \quad (553.3-42)$$

where

$\xi_{web} = (f_1 - f_2)/f_1$, stress gradient in the web, where f_1 and f_2 are the stresses at the opposite ends of the web, $f_1 < f_2$, compression is positive, tension is negative, and the stresses are calculated on the basis of the gross section, (e.g., pure symmetrical bending, $f_1 = f_2$, $\xi_{web} = 2$)

where

$$\xi_{web} = (f_1 - f_2)/f_1, s$$

3. Rational Elastic Buckling Analysis

A rational elastic buckling analysis that considers distortional buckling shall be permitted to be used in lieu of the expressions given in Section 553.3.1.4 shall apply. The safety and resistance factors in Section 553.3.1.4.

553.3.2 Shear

553.3.2.1 Shear Strength of Webs without Holes

The nominal shear strength, V_n , shall be calculated in accordance with Eq. 553.3-43. The safety factor and resistance factors given in this section shall be used to determine the allowable shear strength or design shear strength in accordance with the applicable design method in Section A4, A5, or A6.

$$V_n = A_w F_y \quad (553.3-43)$$

$$\phi_v = 0.95 \text{ (LRFD)} \quad \Omega_v = 1.60 \text{ (ASD)}$$

$$1. \text{ For } \sqrt{h/t} < \sqrt{\frac{Ek_v}{F_y}}$$

$$F_y = 0.60 F_y \quad (553.3-44)$$

$$2. \text{ For } \sqrt{\frac{Ek_v}{F_y}} < h/t \leq 1.51 \sqrt{\frac{Ek_v}{F_y}}$$

$$F_v = \frac{0.60 \sqrt{\frac{Ek_v}{F_y}}}{\left(\frac{h}{t}\right)} \quad (553.3-45)$$

$$3. \text{ For } h/t > 1.51 \sqrt{\frac{Ek_v}{F_y}}$$

$$F_v = \frac{\pi^2 Ek_v}{12(1 - \mu^2) \left(\frac{h}{t}\right)^2} \quad (553.3-46)$$

where

$$\begin{aligned} V_n &= \text{Nominal shear strength [resistance]} \\ A_w &= \text{Area of web element} \\ &= ht \end{aligned} \quad (553.3-47)$$

where

$$\begin{aligned} h &= \text{Depth of flat portion of web measured along plane of web} \\ t &= \text{Web thickness} \\ F_y &= \text{Nominal shear stress} \\ E &= \text{Modulus of elasticity of steel} \end{aligned}$$

k_v = Shear buckling coefficient calculated in accordance with (1) or (2) as follows:

(1) For unreinforced webs, $k_v = 5.34$

(2) For webs with transverse stiffeners satisfying the requirements of Section 553.3.7

When $a/h \leq 1.0$

$$k_v = 4.00 + \frac{5.34}{\left(\frac{a}{h}\right)^2} \quad (553.3-48)$$

When $a/h > 1.0$

$$k_v = 5.34 + \frac{4.00}{\left(\frac{a}{h}\right)^2} \quad (553.3-49)$$

where

$$\begin{aligned} a &= \text{Shear panel length of unreinforced web element} \\ &= \text{Clear distance between transverse stiffeners of reinforced web elements} \\ F_y &= \text{Design yield stress as determined in accordance with Section 551.7.1} \\ \mu &= \text{Poisson's ratio} \\ &= 0.3 \end{aligned}$$

For a web consisting of two or more sheets, each sheet shall be considered as a separate element carrying its share of the shear force.

553.3.2.2 Shear Strength of C-section Webs with Holes

The provisions of this section shall apply within the following limits:

1. $d_h/h \leq 0.7$,
2. $h/t \leq 200$,
3. Holes centered at mid-depth of web,
4. Clear distance between holes ≥ 457 mm,
5. Non-circular holes, corner radii $\geq 2t$,
6. Non-circular holes, $d_h \leq 64$ mm and $L_h \leq 114$ mm,
7. Circular holes, diameter ≤ 152 mm, and

8. $d_h < 14$ mm.

where

- d_h = Depth of web hole
 h = Depth of flat portion of web measured along plane of web
 t = Web thickness
 L_h = Length of web hole

For C-Section webs with holes, the shear strength shall be calculated in accordance with Section 553.3.2.1, multiplied by the reduction factor, q_s , as defined in this section.

$$\text{When } c/t \geq 54 \quad q_s = 1.0$$

$$\text{When } 5 \leq c/t < 54 \quad q_s = c/(54t) \quad (553.3-50)$$

where

$$c = h/2 - d_h/2.83 \text{ for circular holes} \quad (553.3-51)$$

$$= h/2 - d_h/2 \text{ for non-circular holes} \quad (553.3-52)$$

553.3.3 Combined Bending and Shear

553.3.3.1 ASD Method

For beams subjected to combined bending and shear, the required flexural strength, M , and required shear strength, V shall not exceed M_n/Ω_v , respectively.

For beams with unreinforced webs, the required flexural strength, M , and required shear strength, V , shall also satisfy the following interaction equation:

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right) + \left(\frac{\Omega_v V}{V_n}\right)^2} \leq 1.0 \quad (553.3-53)$$

For beams with transverse web stiffeners, when $\Omega_b M/M_{nxo} > 0.5$ and $\Omega_v V/V_n > 0.7$, M and V shall also satisfy the following interaction equation:

$$0.60 \left(\frac{\Omega_b M}{M_{nxo}}\right) + \left(\frac{\Omega_v V}{V_n}\right) \leq 1.30 \quad (553.3-54)$$

where

$$M_n = \text{Nominal flexural strength when bending alone is considered}$$

$$\Omega_b = \text{Safety factor for bending (see Section 553.3.1(a))}$$

$$M_{nxo} = \text{Nominal flexural strength about centroidal x-axis determined in accordance with Section 553.3.1(a)}$$

$$\Omega_v = \text{Safety factor for shear (see Section 553.3.2)}$$

$$V_n = \text{Nominal shear strength when shear alone is considered}$$

553.3.3.2 LRFD Method

For beams subjected to combined bending and shear, the required flexural strength, $\bar{M}$, and the required shear strength, $\bar{V}$, shall not exceed $\phi_b M_n$ and $\phi_v V_n$, respectively.

For beams with unreinforced webs, the required flexural strength, $\bar{M}$, and the required shear strength, $\bar{V}$, shall also satisfy the following interaction equation:

$$\sqrt{\left(\frac{\bar{M}}{\phi_b M_{nxo}}\right)^2 + \left(\frac{\Omega_v \bar{V}}{V_n}\right)^2} \leq 1.0 \quad (553.3-55)$$

For beams with transverse web stiffeners, when $\bar{M}/(\phi_b M_{nxo}) > 0.5$ and $\bar{V}/(\phi_v V_n) > 0.7$, $\bar{M}$ and $\bar{V}$ shall also satisfy the following interaction equation:

$$0.60 \left(\frac{\bar{M}}{\phi_b M_{nxo}}\right) + \left(\frac{\bar{V}}{\phi_v V_n}\right) \leq 1.30 \quad (553.3-56)$$

where

$$M_n = \text{Nominal flexural strength when bending alone is considered}$$

$$M = \text{Required flexural strength}$$

$$= M_u \text{ (LRFD)}$$

$$\phi_b = \text{Resistance factor for bending (see Section 553.3.1a)}$$

$$M_{nxo} = \text{Nominal flexural strength [movement resistance] about centroidal x-axis determined in accordance with Section 553.3.1a}$$

$$V = \text{Required shear strength}$$

$$= V_u \text{ (LRFD)}$$

$$\phi_v = \text{Resistance factor for shear (see Section 553.3.2)}$$

$$V_n = \text{Nominal shear strength when shear alone is considered}$$

553.3.4 Web Crippling

553.3.4.1 Web Crippling Strength of Webs without Holes

The nominal web crippling strength, P_n , shall be determined in accordance with Eq. 553.3-57 or Eq. 553.3-58, as applicable. The safety factors and resistance factors in Tables 553-1 to 553-5 shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$P_n = Ct^2F_y \sin \theta \left(1 - C_R \sqrt{\frac{R}{t}} \right) \left(1 + C_N \sqrt{\frac{N}{t}} \right) \left(1 - C_h \sqrt{\frac{h}{t}} \right) \quad (553.3-57)$$

where

- P_n = Nominal web crippling strength [resistance]
- C = Coefficient from Table 553.3-4(a.1) to 553.3-4(a.5)
- t = Web thickness
- F_y = Design yield stress as determined in accordance with Section 551.7.1
- θ = Angle between plane of web and plane of bearing surface, $45^\circ \leq \theta \leq 90^\circ$
- C_R = Inside bend radius coefficient from Table 553.3-4a.1 to 553.3.4(a.5)
- R = Inside bend radius
- C_N = Bearing length coefficient from Table 553.3-4a.1 to 553.3-4(a.5)
- N = Bearing length 19mm minimum
- C_h = Web slenderness coefficient from Table 553.3-4(a.1) to 553.3-4(a.5)
- h = Flat dimension of web measured in plane of web

Alternatively, for an end-one-flange loading condition on a C- or Z-section, the nominal web crippling strength, P_{nc} shall not be larger than the interior-one-flange loading condition:

$$P_{nc} = \alpha P_n \quad (553.3-58)$$

where

- P_{nc} = Nominal web crippling strength of C and Z-sections with overhang(s)

$$\alpha = \frac{1.34 \left(\frac{L_o}{h} \right)^{0.26}}{0.009 \left(\frac{h}{t} \right) + 0.3} \geq 1.0 \quad (553.3-59)$$

where

- L_o = Overhang length measured from edge of bearing to the end of the member
- P_n = Nominal web crippling strength with end one-flange loading as calculated by 553.3-57 and Tables 553-2 and 553-3 Eq. 553.3-58 and shall be limited to $0.5 \leq L_o/h \leq 1.5$ and $h/t \leq 154$. For L_o/h or h/t outside these limits, $\alpha = 1$.

Webs of members in bending for which h/t is greater than 200 shall be provided with means of transmitting concentrated loads or reactions directly into the web(s).

P_n and P_{nc} shall represent the nominal strengths for load or reaction for one solid web connecting top and bottom flanges. For webs consisting of two or more such sheets, P_n and P_{nc} shall be calculated for each individual sheet and the results added to obtain the nominal strength for the full section.

One-flange loading or reaction shall be defined as the condition where the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is equal to or greater than $1.5h$.

Two-flange loading or reaction shall be defined as the condition where the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is less than $1.5h$.

Table 553-1 shall apply to I-beams made from two channels connected back-to-back where:

$$h/t \leq 200, N/t \leq 210, N/h \leq 1.0 \text{ and } \theta = 90^\circ.$$

See Section 553.3.4.1 of commentary for further explanation.

Table 553-2 shall apply to single web channel and C-Sections members where:

$$h/t \leq 200, N/t \leq 210, N/h \leq 2.0, \text{ and } \theta = 90^\circ.$$

In Table 553-2, for interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of bearing to the end of the member shall be extended at least $2.5h$. For unfastened cases, the distance from the edge of bearing to the end of the member shall be extended at least $1.5h$.

Table 553-3 shall apply to single web Z-section members where $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$, and $\theta = 90^\circ$. In Table 553-3, for interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of bearing to the end of the member shall be extended at least $2.5h$; for unfastened cases, the distance from the edge of bearing to the end of the member shall be extended at least $1.5h$.

Table 553-4 shall apply to single hat section members where $h/t \leq 200$, $N/t \leq 200$, $N/h \leq 2.0$, and $\theta = 90^\circ$.

Table 553-5 shall apply to multi-web section members where $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 3$, and $45^\circ \leq \theta \leq 90^\circ$.

Table 553-1 Safety Factors, Resistance Factors and Coefficients for Built-Up Sections

Support and Flange Conditions	Load Cases			C	C_R	C_N	C_h	ASD Ω_w	LRFD ϕ_w	Limits
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	0.001	2.00	0.75	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	$R/t \leq 5$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	0.001	2.00	0.75	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	$R/t \leq 3$
		Two-Flange Loading or Reaction	End	15.5	0.09	0.08	2.00	2.00	0.75	$R/t \leq 3$
			Interior	36	0.14	0.08	2.00	2.00	0.75	$R/t \leq 3$
	Unstiffened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	2.00	2.00	0.75	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	$R/t \leq 3$

Table 553-2 Safety Factors, Resistance Factors and Coefficients for Single Web Channel and C-Sections

Support and Flange Conditions		Load Cases		C	C_R	C_N	C_h	ASD Ω_w	LRFD ϕ_w	Limits
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.75	0.85	$R/t \leq 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	$R/t \leq 5$
		Two-Flange Loading or Reaction	End	7.5	0.08	0.12	0.048	1.75	0.85	$R/t \leq 12$
			Interior	20	0.10	0.08	0.031	1.75	0.85	$R/t \leq 12$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.85	0.80	$R/t \leq 5$
			Interior	13	0.23	0.14	0.01	1.65	0.90	
		Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	1.65	0.90	$R/t \leq 3$
			Interior	24	0.52	0.15	0.001	1.9	0.80	
Unfastened	Unstiffened Flanges	One-Flange Loading or Reaction	End	4	0.40	0.60	0.03	1.8	0.85	$R/t \leq 2$
			Interior	13	0.32	0.10	0.01	1.8	0.85	$R/t \leq 1$
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	$R/t \leq 1$
			Interior	13	0.47	0.25	0.04	1.90	0.80	

Table 553-3 Safety Factors, Resistance Factors and Coefficients for Single Multi-Web Deck Sections

Support and Flange Conditions		Load Cases		C	C_R	C_N	C_h	ASD Ω_w	LRFD ϕ_w	Limits
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.75	0.85	$R/t \leq 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	$R/t \leq 5.5$
		Two-Flange Loading or Reaction	End	9	0.05	0.16	0.052	1.75	0.85	$R/t \leq 12$
			Interior	24	0.07	0.07	0.04	1.85	0.80	$R/t \leq 12$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	5	0.09	0.02	0.001	1.80	0.85	$R/t \leq 5$
			Interior	13	0.23	0.14	0.01	1.65	0.90	
		Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	1.65	0.90	$R/t \leq 3$
			Interior	24	0.52	0.15	0.001	1.90	0.80	
	Unstiffened Flanges	One-Flange Loading or Reaction	End	4	0.40	0.60	0.03	1.80	0.85	$R/t \leq 2$
			Interior	13	0.32	0.10	0.01	1.80	0.85	$R/t \leq 1$
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	$R/t \leq 1$
			Interior	13	0.47	0.25	0.046	1.90	0.80	

Table 553-4 Safety Factors, Resistance Factors and Coefficients for Single Hat Sections

Support and Flange Conditions	Load Cases		C	C_R	C_N	C_h	ASD Ω_w	LRFD ϕ_w	Limits
Fastened to Support	One-Flange Loading or Reaction	End	4	0.25	0.68	0.04	2.00	0.75	$R/t \leq 5$
		Interior	17	0.13	0.13	0.04	1.80	0.85	$R/t \leq 10$
	Two-Flange Loading or Reaction	End	9	0.10	0.07	0.03	1.75	0.85	$R/t \leq 10$
		Interior	10	0.14	0.22	0.02	1.80	0.85	
Unfastened	One-Flange Loading or Reaction	End	4	0.25	0.68	0.04	2.00	0.75	$R/t \leq 4$
		Interior	17	0.13	0.13	0.04	1.80	0.85	$R/t \leq 4$

Table 553-5 Safety Factors, Resistance Factors and Coefficients for Multi-Web Deck Sections

Support and Flange Conditions	Load Cases		C	C_R	C_N	C_h	ASD Ω_w	LRFD ϕ_w	Limits
Fastened to Support	One-Flange Loading or Reaction	End	4	0.04	0.25	0.025	1.70	0.90	$R/t \leq 20$
		Interior	8	0.10	0.17	0.004	1.75	0.85	$R/t \leq 10$
	Two-Flange Loading or Reaction	End	9	0.12	0.14	0.040	1.80	0.85	$R/t \leq 10$
		Interior	10	0.11	0.21	0.020	1.75	0.85	
Unfastened	One-Flange Loading or Reaction	End	3	0.04	0.29	0.028	2.45	0.60	$R/t \leq 20$
		Interior	8	0.10	0.17	0.004	1.75	0.85	
	Two-Flange Loading or Reaction	End	6	0.16	0.15	0.050	1.65	0.90	$R/t \leq 5$
		Interior	17	0.10	0.10	0.046	1.65	0.90	

553.3.4b Web Crippling Strength of C-Section Webs with Holes

Where a web hole is within the bearing length, a bearing stiffener shall be used. For beam webs with holes, the available web crippling strength shall be calculated in accordance with Section 553.3.4(a), multiplied by the reduction factor, R_c , given in this section.

The provisions of this section shall apply within the following limits:

1. $d_h/h \leq 0.7$,
2. $h/t \leq 200$,
3. Hole centered at mid-depth of web,
4. Clear distance between holes ≥ 457 mm,
5. between end of member and edge of hole $\geq d$,
6. Non-circular holes, corner radii $\geq 2t$,
7. Non-circular holes, $d_h \leq 64$ mm and $L_h \leq 114$ mm,
8. Circular holes, $d_h \leq 152$ mm, and
9. $d_h > 14$ mm.

where

- d_h = Depth of web hole
 h = Depth of flat portion of web measured along plane of web
 t = Web thickness
 d = Depth of cross-section
 L_h = Length of web hole

For end-one flange reaction (Eq. C3.4.1-1 with Table 553.3-4a.2) where a web hole is not within the bearing length, the reduction factor, R_c , shall be calculated as follows:

$$R_c = 1.01 - 0.325d_h/h + 0.083x/h \leq 1.0 \quad (553.3-59)$$

$$N \geq 75 \text{ mm}$$

where

- x = Nearest distance between web hole and edge of bearing
 N = Bearing length

553.3.5 Combined Bending and Web Crippling

553.3.5.1 ASD Method

Unreinforced flat webs of shapes subjected to a combination of bending and concentrated load or reaction shall be designed such that the moment, M , and the concentrated load or reaction, P , satisfy $M \leq M_{nxo}/\Omega_b$, and $P \leq P_n/\Omega_w$. In addition, the following requirements in (a), (b) and (c), as applicable, shall be satisfied.

1. For shapes having single unreinforced webs, Eq. 553.3-60 shall be satisfied as follows:

$$0.91 \left(\frac{P}{P_n} \right) + \left(\frac{M}{M_{nxo}} \right) \leq \frac{1.33}{\Omega} \quad (553.3-60)$$

Exception:

At the interior supports of continuous spans, Eq. 553.3-60 shall not apply to deck or beams with two or more single webs, provided the compression edges of adjacent webs are laterally supported in the negative moment region by continuous or intermittently connected flange elements, rigid cladding, or lateral bracing, and the spacing between adjacent webs does not exceed 254 mm,

2. For shapes having multiple unreinforced webs such as I-sections made of two C-sections connected back-to-back, or similar sections that provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a C-section), Eq. 553.3-61 shall be satisfied as follows:

$$0.88 \left(\frac{P}{P_n} \right) + \left(\frac{M}{M_{nxo}} \right) \leq \frac{1.46}{\Omega} \quad (553.3-61)$$

3. For the support point of two nested Z-shapes, Eq. 553.3-62 shall be satisfied as follows:

$$0.86 \left(\frac{P}{P_n} \right) + \left(\frac{M}{M_{nxo}} \right) \leq \frac{1.65}{\Omega} \quad (553.3-62)$$

Eq. 553.3-62 shall apply to shapes that meet the following limits:

$$h/t \leq 150,$$

$$N/t \leq 140,$$

$$F_y \leq 483 \text{ Mpa, and}$$

$$R/t \leq 5.5$$

The following conditions shall also be satisfied:

- The ends of each section are connected to the other section by a minimum of two 12.7 mm diameter A307 bolts through the web.
- The combined section is connected to the support by a minimum of two 12.7 mm diameter A307 bolts through the flanges.
- The webs of the two sections are in contact.

- d. The ratio of the thicker to the thinner part does not exceed 1.3.

The following conditions shall be satisfied;

M	= Required flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction, P
P	= Required strength for concentrated load or reaction in the presence of bending moment
M_{nxo}	= Nominal flexural strength about centroidal x-axis determined in accordance with Section 553.3.1.1
Ω_b	= Safety factor for bending (See Section 553.3.1.1)
P_n	= Nominal strength for concentrated load or reaction in absence of bending moment determined in accordance with Section 553.3.4
Ω_w	= Safety factor for web crippling (See Section 553.3.4)
Ω	= Safety factor for combined bending and web crippling. = 1.70.

553.3.5b LRFD Methods

Unreinforced flat webs of shapes subjected to a combination of bending and concentrated load or reaction shall be designed such that the moment, M , and the concentrated load or reaction, $\bar{P}$, satisfy $\bar{M} \leq \phi_b M_{nxo}$ and $\bar{P} \leq \phi_w P_n$. In addition, the following requirements in (a), (b), (c), as applicable, shall be satisfied.

1. For shapes having single unreinforced webs, Eq. 553.3-63 shall be satisfied as follows:

$$0.91 \left(\frac{\bar{P}}{P_n} \right) + \left(\frac{\bar{M}}{M_{nxo}} \right) \leq 1.33\phi \quad (553.3-63)$$

where

$$\phi = 0.90(\text{LRFD})$$

Exception:

At the interior supports of continuous spans, Eq. 553.3-62 shall to deck or beams with two or more single webs, provided the compression edges of adjacent webs are laterally supported in the negative moment region by continuous or intermittently connected flange elements, rigid cladding, or lateral bring, and the spacing between adjacent webs does not exceed 254 mm.

2. For having multiple unreinforced webs such as I-Sections made of two C-sections connected back-to-back, or similar sections that provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a C-section), Eq. 553.3-63a shall be satisfied as follows:

$$0.88 \left(\frac{\bar{P}}{P_n} \right) + \left(\frac{\bar{M}}{M_{nxo}} \right) \leq 1.46\phi \quad (553.3-63a)$$

where

$$\phi = 0.90 \text{ (LRFD)}$$

3. For two nested Z-shapes, Eq. 553.3-64 shall be satisfied as follows:

$$0.88 \left(\frac{\bar{P}}{P_n} \right) + \left(\frac{\bar{M}}{M_{nxo}} \right) \leq 1.65\phi \quad (553.3-64)$$

where

$$\phi = 0.90 \text{ (LRFD)}$$

Eq. 553.3-64 shall apply to shapes that meet the following limits:

$$h/t \leq 150,$$

$$N/t \leq 140,$$

$$F_y \leq 483 \text{ Mpa, and}$$

$$R/t \leq 5.5$$

The following conditions shall also be satisfied:

- The ends of each section are connected to the other section by a minimum of two 12.7 mm diameter A307 bolts through the web.
- The combined section is connected to the support by a minimum of two 12.7 mm diameter A307 bolts through flanges.
- The webs of the two sections are in contact.
- The ration of the thicker to the thinner part does not exceed 1.3.

The following notation shall apply in this section:

- $\bar{M}$ = Required flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction, $\bar{P}$
- $\bar{P}$ = M_u (LRFD)
- $\bar{P}$ = required strength for concentrated load or reaction [factored concentrated load or reaction in presence to bending moment]
- $\bar{P}_u$ (LRFD)
- ϕ_b = resistance factor for bending (See Section 553.3.1.1)
- M_{nxo} = Nominal flexural strength about centroidal x-axis determined in accordance with Section 553.3.1.1
- ϕ_w = Resistance factor for web crippling (See Section 553.3.4)
- P_n = Nominal strength for concentrated load or reaction in absence of bending moment determined in accordance with Section 553.3.4

553.3.6 Combined Bending and Torsional Loading

For laterally unrestrained flexural members subjected to both bending and torsional loading, the available flexural strength [factored moment resistance] calculated in accordance with Section 553.3.1a (a) shall be reduced by multiplying it by a reduction factor, R .

As specified in Eq. 553.3-65, the reduction factor, R , shall be equal to the ratio of the normal stresses due to bending alone divided by the combined stresses due to both bending and torsional warping at the point of maximum combined stress on the cross-section.

$$R = \frac{f_{bending}}{f_{bending} + f_{torsion}} \leq 1 \quad (553.3-65)$$

Stresses shall be calculated using full section properties for the torsional stresses and effective section properties for the bending stresses. For C-sections with edge stiffened flanges, if the maximum combined compressive stresses occur at the junction of the web and flange, the R factor shall be permitted to be increased by 15 percent. But the R factor shall not be greater than 1.0

The provisions of this section shall not be applied when the provisions of Section shall not be applied when the provisions of Section 554.6.1a and 554.6.1b are used.

553.3.7 Stiffeners

553.3.7a Bearing Stiffeners

Bearing Stiffeners attached to beam webs at points of concentrated loads or reactions shall be designed as compression members. Concentrated loads or reactions shall be applied directly into the stiffeners or each stiffener shall be fitted accurately to the flat portion of the flange to provide direct load bearing into the end of the stiffener. Means for shear transfer between the stiffener and the web shall be provided in accordance with Section 555. For concentrated loads or reactions, the nominal strength, P_n , shall be the smaller value calculated by (a) and (b) of this section. The safety factor and resistance factors provided in this section shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section 551.4, or 551.5.

$$\phi_c = 0.85 \text{ (LRFD)} \quad \Omega_c = 2.00 \text{ (ASD)}$$

$$1. \quad P_n = F_{wy} A_c \quad (553.3-66)$$

$$2. \quad P_n = \text{Nominal axial strength [resistance] evaluated in accordance with Section 553.4.1 (a), with } A_e \text{ replaced by } A_b$$

where

F_{wy} = Lower value of f_y for beam web, or F_{ys} for stiffener section

$$\begin{aligned} A_c &= 18t^2 + A_s \text{ for bearing stiffener at interior support or under concentrated load} \quad (553.3-67) \\ &= 10t^2 + A_s \text{ for bearing stiffener at end support} \quad (553.3-68) \end{aligned}$$

where

$$\begin{aligned} t &= \text{Base steel thickness of beam web} \\ A_s &= \text{Cross-sectional area of bearing stiffener} \\ A_b &= b_1 t + A_s, \text{ for bearing stiffener at interior support or under concentrated load} \quad (553.3-69) \\ &= b_2 t + A_s, \text{ for bearing stiffener at end support} \quad (553.3-70) \end{aligned}$$

where

$$b_1 = 25t[0.0024(L_{st}/t) + 0.72] \leq 25t \quad (553.3-71)$$

$$b_2 = 12t[0.0044(L_{st}/t) + 0.83] \leq 12t \quad (553.3-72)$$

where

$$L_{st} = \text{Length of bearing stiffener}$$

The w/t_s ratio for the stiffened and unstiffened elements of the bearing stiffener shall not exceed $1.28 \sqrt{E/F_{ys}}$ and

$0.42 \sqrt{E/F_{ys}}$, respectively, where F_{ys} is the yield stress, and t_s is the thickness of the stiffener steel.

553.3.7b Bearing Stiffeners in C-Section Flexural Members

For two-flange loading of C-section flexural members with bearing stiffeners that do not meet the requirements of Section 553.3.7a, the nominal strength [resistance], P_n , shall be calculated in accordance with Eq. 553.3-73. The safety factor and resistance factors in this section shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.6.

$$P_n = 0.7(P_{wc} + A_e F_y) \geq P_{wc} \quad (553.3-73)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.70 \text{ (ASD)}$$

where

P_{wc} = Nominal web crippling strength [resistance] for C-section flexural member calculated in accordance with Eq. 553.3-57 for single web members, at end or interior locations

A_e = Effective area of bearing stiffener subjected to uniform compressive stress, calculated at yield stress

F_y = Yield stress of bearing stiffener steel Eq. 553.3-73 shall apply within the following limits:

1. Full bearing of the stiffener is required. If the bearing width is narrower than the stiffener such that one of the stiffener flanges is unsupported, P_n is reduced by 50 percent.
2. Stiffeners are C-section stud or track members with minimum web depth of 89 mm and a minimum base steel thickness of 0.84 mm.
3. The stiffener is attached to the flexural member web with at least three fasteners (screw or bolts).
4. The distance from the flexural member flanges to the first fastener (s) is not less than $d/8$, where d is the overall depth of the flexural member.
5. The length of the stiffener is not less than the depth of the flexural member minus 9 mm.

6. The bearing width is not less than 38 mm.

553.3.7c Shear Stiffener

Where shear stiffeners are required, the spacing shall be based on the nominal shear strength, V_n , permitted by Section 553.3.2, and the ratio a/h shall not exceed $[260 / (h/t)]^2$ nor 3.0.

The actual moment of inertia, I_s , of a pair of attached shear stiffeners, or of a single shear stiffener, with reference to an axis in the plane of the web, shall have a minimum value calculated in accordance with Eq. 553.3-73 as follows:

$$I_{smin} = 5ht^3[h/a - 0.7(a/h)] \geq (h/50)^4 \quad (553.3-73)$$

where

h and t = Values as defined in Section 552.1.2

a = Distance between shear stiffeners

The gross area of shear stiffeners shall not be less than:

$$A_{st} = \frac{1 - C_v}{2} \left[\left(\frac{a}{h} \right) - \frac{\left(\frac{a}{h} \right)^2}{\left(\frac{a}{h} \right) + \sqrt{1 + \left(\frac{a}{h} \right)^2}} \right] YDht \quad (553.3-74)$$

where

$$C_v = \frac{1.53Ek_v}{F_y \left(\frac{h}{t} \right)^2} \quad \text{when } C_v \leq 0.80$$

$$= \frac{1.11}{\left(\frac{h}{t} \right)} \sqrt{\frac{Ek_v}{F_y}} \quad \text{when } C_v > 0.80 \quad (553.3-75)$$

where

$$k_v = 4.00 + \frac{5.34}{\left(\frac{a}{h} \right)^2} \quad \text{when } a/h \leq 1.0 \quad (553.3-76)$$

$$= 5.34 + \frac{4.00}{\left(\frac{a}{h} \right)^2} \quad \text{when } a/h > 1.0 \quad (553.3-77)$$

where

$$Y = \frac{\text{Yield stress web steel}}{\text{Yield stress stiffener steel}}$$

$$D = \begin{aligned} &1.0 \text{ for stiffeners furnished in pairs} \\ &= 1.8 \text{ for single-angle stiffeners} \\ &= 2.4 \text{ for single-plate stiffeners} \end{aligned}$$

553.3.7d Non-Conforming Stiffeners

The available strength of members with stiffeners that do not meet the requirements of Section 553.3.7.1, 553.3.7.2, or 553.3.7.3, such as stamped or rolled-in stiffeners, shall be determined by tests in accordance with Section 556 or rational engineering analysis in accordance with Section 551.1.2 (b).

553.4 Concentrically Loaded Compression Members

The available axial strength shall be the smaller of the values calculated in accordance with Sections 553.4.1, 553.4.2, 554.1.2, 554.1.2, 554.6.1.3 and 554.6.1.4, where applicable.

553.4.1 Nominal Strength for Yielding, Flexural-Torsional and Torsional Buckling

This section shall apply to members in which the resultant of all loads acting on the member is an axial load passing through the centroid of the effective section calculated at the stress, F_n , defined in this section.

1. The nominal axial strength, P_n , shall be calculated in accordance with Eq. 553.4-1. The safety factor and resistance factors in this section shall be used to determine the allowable axial strength or design axial strength in accordance with the applicable design method in Section 551.4, 551.5.

$$P_n = A_e F_n \quad (553.4-1)$$

$$\phi_c = 0.85 \text{ (LRFD)} \quad \Omega_c = 1.80 \text{ (ASD)}$$

where

A_e = Effective area calculated at stress F_n . For sections with circular holes, A_e is determined from the effective width in accordance with Section 552.2.2 (a), subject to the limitations of that section. If the number of holes in the effective length region times the hole diameter divided by the effective length does not exceed 0.015, it is permitted to determine A_e by ignoring the holes. For closed cylindrical tubular members, A_e is provided in Section 553.4.1.5.

a. F_n shall be calculated as follows:

For $\lambda_c \leq 1.5$

$$F_n = (0.658\lambda_c^2)F_y \quad (553.4-2)$$

For $\lambda_c > 1.5$

$$F_n = \left[\frac{0.877}{\lambda_c^2} \right] F_y \quad (553.4-3)$$

where

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \quad (553.4-4)$$

F_e = The least of the applicable elastic flexural, torsional and flexural-torsional buckling stress determined in accordance with Sections 553.4.1.1 through 553.4.1.5

2. Concentrically loaded angle sections shall be design for an additional bending moment as specified in the definitions of M_x and M_y (ASD) or M_x and M_y (LRFD or LSD) in Section 553.5.2.

553.4.1.1 Sections Not Subject to Torsional or Flexural-Torsional Buckling

For doubly-symmetric sections, closed cross-sections, and any other sections that can be shown not to be subjected to torsional or flexural-torsional buckling, the elastic flexural buckling stress, F_e , shall be calculated as follows:

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r} \right)^2} \quad (553.4-5)$$

where

- E = Modulus of elasticity of steel
- K = Effective length factor
- L = Laterally unbraced length of member
- r = Radius of gyration of full unreduced cross section about axis of buckling

In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or floor slabs or roof deck secured horizontally by walls or bracing systems parallel to the plane of the frame, and in trusses, the effective length factor, K , for compression members that do not depend upon their own bending stiffness for lateral stability of the frame or truss shall be taken as unity, unless analysis shows

that a smaller value is suitable. In a frame that depends upon its own bending stiffness for lateral stability, the effective length, KL , of the compression members shall be determined by a rational method and shall not be less than the actual unbraced length.

553.4.1.2 Doubly or Singly-symmetric Sections Subject to Torsional or Flexural-Torsional Buckling

For singly-symmetric sections subject to flexural-torsional buckling, F_e shall be taken as the smaller of F_e calculated in accordance with Section 553.4.1.(d) and F_e calculated as follows:

$$F_e = \frac{1}{2\beta} \left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t} \right] \quad (553.4-6)$$

Alternatively, a conservative estimate of F_e shall be permitted to be calculated as follows:

$$F_e = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} \quad (553.4-7)$$

where

$$\beta = 1 - (x_o/r_o)^2 \quad (553.4-8)$$

σ_t and σ_{ex} = Values as defined in Section 553.3.1b.1

For singly-symmetric sections, the x-axis shall be selected as the axis of symmetry.

For doubly-symmetric sections subject to torsional buckling, F_e shall be taken as the smaller of F_e calculated in accordance with Section 553.4.1.1 and $F_e = \sigma_t$, where σ_t is defined in Section 553.3.1b.1.

For singly-symmetric unstiffened angle sections for which the effective area (A_e) at stress F_y is equal to the full unreduced cross-sectional area (A), F_e shall be computed using Eq.553.4-5 where is the least radius of gyration.

553.4.1.3 Point-Symmetric Sections

For point-symmetric sections, F_e shall be taken as the lesser of σ_t as defined in Section 553.3.1b.1 and F_e as calculated in Section 553.4.1.1 using the minor principal axis of the section.

553.4.1.4 Nonsymmetric Sections

For shapes whose cross-sections do not have any symmetry, either about an axis or about a point, F_e shall be determined by rational analysis. Alternatively, compression members composed of such shapes shall be permitted to be tested in accordance with Section 556.

553.4.1.5 Closed Cylindrical Tubular Sections

For closed cylindrical tubular members having a ratio of outside diameter to wall thickness, D/t , not greater than $0.441E/F_y$ and in which the resultant of all loads and moments acting on the member is equivalent to a single force in the direction of the member axis passing through the centroid of the section, the elastic flexural buckling stress, F_e shall be calculated in accordance with Section 553.4.1(a), and the effective area, A_e , shall be calculated as follows:

$$A_e = A_o + R(A - A_o) \quad (553.4-9)$$

where

$$A_o = \left[\frac{0.037}{\frac{DF_y}{tE}} + 0.667 \right] A \leq A$$

for $D/t \leq 0.441E/F_y$ (553.4-10)

where

$$\begin{aligned} D &= \text{Outside diameter of cylindrical tube} \\ F_y &= \text{Yield stress} \\ t &= \text{Thickness} \\ E &= \text{Modulus of elasticity of steel} \\ A &= \text{Area of full unreduced cross-section} \\ R &= F_y(2F_e) \leq 1.0 \end{aligned} \quad (553.4-11)$$

553.4.2 Distortional Buckling Strength

The provisions of this section shall apply to *I*-, *Z*-, *C*-, *Hat*, and other open cross section members that employ flanges with edge stiffeners, with the exception of members that are designed in accordance with Section 554.6.1.2. The nominal axial strength shall be calculated in accordance with Eqs. C4.2-1 and C4.2-2. The safety factor and resistance factors

In this section shall be used to determine the allowable compressive strength or design compressive strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$\phi_b = 0.85 \text{ (LRFD)} \quad \Omega_c = 1.80 \text{ (ASD)}$$

For $\lambda_d \leq 0.561$

$$P_n = P_y \quad (553.4-12)$$

$$\text{For } P_n = \left(1 - 0.25 \left(\frac{P_{crd}}{P_y} \right)^{0.6} \right) \left(\frac{P_{crd}}{P_y} \right)^{0.6} P_y \quad (553.4-13)$$

where

$$\lambda_d = \sqrt{\frac{P_y}{P_{crd}}} \quad (553.4-14)$$

P_n = Nominal axial strength

$$P_y = A_g F_y \quad (553.4-15)$$

where

A_g = Gross area of the cross-section
 F_y = Yield stress

$$P_{crd} = A_g F_d \quad (553.4-16)$$

where

F_d = Elastic distortional buckling stress calculated in accordance with either Section 553.4.2(a), (b), or (c)

1. Simplified Provision for Unrestrained C- and Z-Sections with simple Lip Stiffeners

For C- and Z-Sections that have no rotational restraint of the flange and that are within the dimensional limits provided in this Section 553 shall be permitted to be used to calculate a conservative prediction of distortional buckling stress, F_d . See Section 553.4.2(b) or 553.4.2(c) for alternative options for members outside the dimensional limits.

The following dimensional limits shall apply:

- a. $50 \leq h_o/t \leq 200$,
- b. $25 \leq b_o \leq 100$,
- c. $6.25 < D/t \leq 50$,
- d. $45^\circ \leq \theta \leq 90^\circ$,
- e. $2 \leq h_o/b_o \leq 8$, and
- f. $0.04 \leq D \sin \theta / b_o \leq 0.5$

where

- h_o = Out-to-out web depth as defined in Figure 552-4
 b_o = Out-to-out flange width as defined in Figure 552-4
 D = Out-to-out lip dimension as defined in Figure 552-9
 t = Base steel thickness
 θ = Lip angle as defined in Figure 552-9

The distortional buckling stress, F_d , shall be calculated in accordance with Eq.C4.2-6:

$$F_d = \alpha k_d \frac{\pi^2 E}{12(1 - \mu^2)} \left(\frac{t}{b_o} \right)^2 \quad (553.4-17)$$

where

- α = A value that accounts for the benefit of an unbraced length, L_m , shorter than L_{cr} , but can be conservatively taken as 1.0
 = 1.0 for $L_m \geq L_{cr}$
 = $(L_m/L_{cr})^{1n(L_m/L_{cr})}$ for $L_m < L_{cr}$ (553.4-18)

where

- L_m = Distance between discrete restraints that restrict distortional buckling (for continuously restrained members $L_m = L_{cr}$ but the restraint can be included as a rotational spring, k_ϕ , in accordance with the provisions in 553.4.2(b) or (c)

$$L_{cr} = 1.2 h_o \left(\frac{b_o D \sin \theta}{h_o t} \right)^{0.6} \leq 10 h_o \quad (553.4-19)$$

$$k_d = 0.05 \leq 0.1 \left(\frac{b_o D \sin \theta}{h_o t} \right)^{1.4} \leq 8.0 \quad (553.4-20)$$

E = Modulus of elasticity of steel

μ = Poisson's ratio

2. For C- and Z-Sections or Hat Sections or any Open Section with Stiffened Flanges of Equal Dimension where the Stiffener is either a Simple Lip or a Complex Edge Stiffener

The provisions of this section shall apply to any open section with stiffened flanges of equal dimension, including those meeting the geometric limits of 553.4.2a.

$$F_d = \frac{k_{\phi fe} + k_{\phi we} + k_\phi}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \quad (553.4-21)$$

where

$k_{\phi fe}$ = Elastic rotational stiffness provided by the flange to the flange/web juncture, in accordance with Eq. 553.3-39

$k_{\phi we}$ = Elastic rotational stiffness provided by the web to the flange/web juncture

$$= \frac{Et^3}{6h_o(1 - \mu^2)} \quad (553.4-22)$$

k_ϕ = Rotational stiffness provided by restraining elements (brace, panel, sheathing) to the flange/web juncture of a member (zero if the flange is unrestrained). If rotational stiffness provided to the two flanges is dissimilar, the smaller rotational stiffness is used

$\tilde{k}_{\phi fg}$ = Geometric rotational stiffness (divided by the stress F_d) demanded by the flange from the flange/web juncture, in accordance with Eq. 553.3-41

$\tilde{k}_{\phi wg}$ = Geometric rotational stiffness (divided by the stress F_d) demanded by the web flange/web juncture

$$= \left(\pi/L \right)^2 \left(\frac{th_o^3}{60} \right) \quad (553.4-23)$$

L = Minimum of L_{cr} and L_m

where

$$L_{cr} = \left(\frac{6\pi^4 h_o (1 - \mu^2)}{t^3} \left(I_{xf}(x_o - h_x)^2 + C_{wf} + \frac{I_{xyf}^2}{I_{yf}} (x_o - h_x)^2 \right) \right)^{1/4} \quad (553.4-24)$$

L_m = Distance between discrete restraints that restrict distortional buckling (for continuously restrained members $L_m = L_{cr}$)

See Section 553.3.1.4 (b) for definition of variables in Eq. 553.4-24.

3. Rational Elastic Buckling Analysis

A rational elastic buckling analysis that considers distortional buckling shall be permitted to be used in lieu of the expressions given in Section 553.4.2(a) or (b) the safety and resistance factors in Section 553.4.2 shall apply.

553.5 Combined Axial Load and Bending

553.5.1 Combined Tensile Axial Load and Bending

553.5.1a ASD Method

The required strengths T , M_x , and M_y , shall satisfy the following interaction equations:

$$\frac{\Omega_b M_x}{M_{nxt}} + \frac{\Omega_b M_y}{M_{nyt}} + \frac{\Omega_t T}{T_n} \leq 1.0 \quad (553.5-1)$$

and

$$\frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} + \frac{\Omega_t T}{T_n} \leq 1.0 \quad (553.5-2)$$

where

$$\begin{aligned} \Omega_b &= 1.67 \\ M_x, M_y &= \text{Required flexural strengths with respect to centroidal axes of section} \\ M_{nxt}, M_{nyt} &= S_{ft} F_y \end{aligned} \quad (553.5-3)$$

where

$$\begin{aligned} S_{ft} &= \text{Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis} \\ F_y &= \text{Design yield stress determined in accordance with Section 551.7.1} \\ \Omega_t &= 1.67 \\ T &= \text{Required tensile axial strength} \\ T_n &= \text{Nominal tensile axial strength determined in accordance with Section 553.2} \end{aligned}$$

M_{nx}, M_{ny} = Nominal flexural strengths about centroidal axes determined in accordance with Section 553.3.1

553.5.1b LRFD Method

The required strengths $\bar{T}$, $\bar{M}_x$, and $\bar{M}_y$ shall satisfy the following interaction equations:

$$\frac{\bar{M}_x}{\phi_b M_{nxt}} + \frac{\bar{M}_y}{\phi_b M_{nyt}} + \frac{\bar{T}}{\phi_t T_n} \leq 1.0 \quad (553.5-4)$$

$$\frac{\bar{M}_x}{\phi_b M_{nx}} + \frac{\bar{M}_y}{\phi_b M_{ny}} + \frac{\bar{T}}{\phi_t T_n} \leq 1.0 \quad (553.5-5)$$

where

$$\begin{aligned} \bar{M}_x, \bar{M}_y &= \text{Required flexural strengths [factored moments] with respect to centroidal axes} \\ \bar{M}_x &= M_{ux}, \bar{M}_y = M_{uy} \text{ (LRFD)} \\ \phi_b &= \text{For flexural strength (Section 553.3.1.1),} \\ &\phi_b = 0.90 \text{ or } 0.95 \text{ (LRFD)} \\ &\text{For laterally unbraced beams (Section 553.3.1.2), } \phi_b = 0.90 \text{ (LRFD)} \\ &\text{For closed cylindrical tubular members (Section 553.3.1.3), } \phi_b = 0.90 \text{ (LRFD)} \end{aligned}$$

$$M_{nxt}, M_{nyt} = S_{ft} F_y \quad (553.5-6)$$

where

$$\begin{aligned} S_{ft} &= \text{Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis} \\ F_y &= \text{Design yield stress determined in accordance with Section 551.7.1} \\ T &= \text{Required tensile axial strength} \\ &= T_u \text{ (LRFD)} \\ \phi_t &= 0.95 \text{ (LRFD)} \\ T_n &= \text{Nominal tensile axial strength determined in accordance with Section 553.2} \\ M_{nx}, M_{ny} &= \text{Nominal flexural strengths about centroidal axes determined in accordance with Section 553.3.1} \end{aligned}$$

553.5.2 Combined Compressive Axial Load and Bending

553.5.2a ASD Method

The required strengths P , M_x , and M_y shall be determined using first order elastic analysis and shall satisfy the following interaction equations. Alternatively, the required strengths P , M_x , and M_y shall be determined in accordance with Section C-2 and shall satisfy the following interaction equations using the values for K_x , K_y , a_x , a_y , C_{mx} , and C_{my} specified in Section C-2. In addition, each individual ratio in Eqs. 553.5-4 to 553.5-6 shall not exceed unity.

For singly-symmetric unstiffened angle sections with unreduced effective area, M_y shall be permitted to be taken as the required flexural strength only. For other angle sections or singly-symmetric unstiffened angles for which the effective area (A_e) at stress F_y is less than the full unreduced cross-sectional area (A), M_y shall be taken either as the required flexural strength or the required flexural strength plus $PL/1000$, which results in a lower permissible value of P .

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx} a_x} + \frac{\Omega_b C_{my} M_y}{M_{ny} a_y} \leq 1.0 \quad (553.5-7)$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0 \quad (553.5-8)$$

When $\Omega_c P/P_n \leq 0.15$, the following equation shall be permitted to be used in lieu of the above two equations:

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0 \quad (553.5-9)$$

where

- Ω_c = 1.80
- P = Required compressive axial strength
- P_n = Nominal axial strength determined in accordance with Section 553.4
- Ω_b = 1.67
- M_x, M_y = Required flexural strengths with respect to centroidal axes of effective section determined for required compressive axial strength alone
- M_{nx}, M_{ny} = Nominal flexural strengths about centroidal axes determined in accordance with Section 553.3.1

$$a_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \quad (553.5-10)$$

$$a_y = 1 - \frac{\Omega_c P}{P_{Ey}} > 0 \quad (553.5-11)$$

where

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \quad (553.5-12)$$

$$P_{Ey} = \frac{\pi^2 EI_y}{(K_y L_y)^2} \quad (553.5-13)$$

where

- I_x = Moment of inertia of full unreduced cross-section about x-axis
- K_x = Effective length factor for buckling about x-axis
- L_x = Unbraced length for bending about x-axis
- I_y = Moment of inertia of full unreduced cross-section about y-axis
- K_y = Effective Length factor for buckling about y-axis
- L_y = Unbraced length for bending about y-axis
- P_{no} = Nominal axial strength determined in accordance with Section 553.4, with $F_n = F_y$
- C_{mx}, C_{my} = Coefficients whose values are determined in accordance with (a), (b), or (c) as follows:

- a. For compression members in frames subject to joint translation (sideways)

$$C_m = 0.85$$

- b. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

$$C_m = 0.6 - 0.4(M_1/M_2) \quad (553.5-14)$$

where

- M_1/M_2 = Ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

- c. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m is to be determined by rational analysis. However, in lieu of such analysis, the following values are permitted to be used:

- (1) For members whose ends are restrained,

$$C_m = 0.85, \text{ and}$$

- (2) For members whose ends are unrestrained,

$$C_m = 1.0.$$

553.5.2b LRFD Method

The required strengths $\bar{P}$, $\bar{M}_x$, and $\bar{M}_y$ shall be determined using first order elastic analysis and shall satisfy the following interaction equations. Alternatively, the required strengths $\bar{P}$, $\bar{M}_x$, and $\bar{M}_y$ shall be determined in accordance with Section C-2 and shall satisfy the following interaction equations using the values for K_x , K_y , a_x , a_y , C_{mx} , and C_{my} specified in Section C-2. In addition, each individual ratio in Eqs. 553.5-7 to 553.5-9 shall not exceed unity.

For singly-symmetric unstiffened angle sections with unreduced effective area, $\bar{M}_y$ shall be permitted to be taken as the required flexural strength [factored moment] only. For other angle sections or singly-symmetric unstiffened angles for which the effective area (A_e) at stress F_y is less than the full unreduced cross-sectional area (A), $\bar{M}_y$ shall be taken either as the required flexural strength or the required flexural strength plus $(\bar{P})L/1000$, whichever results in a lower permissible value of $\bar{P}$.

$$\frac{\bar{P}}{\phi_c P_n} + \frac{C_{mx} \bar{M}_x}{\phi_b M_{nx} a_x} + \frac{C_{my} \bar{M}_y}{\phi_b M_{ny} a_y} \leq 1.0 \quad (553.5-15)$$

$$\frac{\bar{P}}{\phi_c P_{no}} + \frac{\bar{M}_x}{\phi_b M_{nx}} + \frac{\bar{M}_y}{\phi_b M_{ny}} \leq 1.0 \quad (553.5-16)$$

When $\bar{P}/\phi_c P_n \leq 0.15$, the following equation shall be permitted to be used in lieu of the above two equations:

$$\frac{\bar{P}}{\phi_c P_n} + \frac{\bar{M}_x}{\phi_b M_{nx}} + \frac{\bar{M}_y}{\phi_b M_{ny}} \leq 1.0 \quad (553.5-17)$$

where

- $\bar{P}$ = Required compressive axial strength
 = P_u (LRFD)
 ϕ_c = 0.85 (LRFD)
 P_n = Nominal axial strength [resistance] determined in accordance with Section 553.4
 $\bar{M}_x, \bar{M}_y$ = Required flexural strengths with respect to centroidal axes of effective section determined for required compressive axial strength alone.

- $\bar{M}_x = M_{ux}, \bar{M}_y = M_{uy}$ (LRFD)
 ϕ_b = For flexural strength (Section 553.3.1.1), $\phi_b = 0.90$ or 0.95 (LRFD)
 = For laterally unbraced flexural members (Section 553.3.1.2), $\phi_b = 0.90$ (LRFD)
 = For closed cylindrical tubular member (Section 553.3.1.3), $\phi_b = 0.95$ (LRFD) and 0.90 (LSD)

M_{nx}, M_{ny} = Nominal flexural strengths about centroidal axes determined in accordance with Section 553.3.1

$$a_x = 1 - \frac{\bar{P}}{P_{Ex}} > 0 \quad (553.5-18)$$

$$a_y = 1 - \frac{\bar{P}}{P_{Ey}} > 0 \quad (553.5-19)$$

where

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \quad (553.5-20)$$

$$P_{Ey} = \frac{\pi^2 EI_y}{(K_y L_y)^2} \quad (553.5-21)$$

where

- I_x = Moment of inertia of full unreduced cross-section about x-axis
 K_x = Effective length factor for buckling about x-axis
 L_x = Unbraced length for bending about x-axis
 I_y = Moment of inertia of full unreduced cross-section about y-axis
 K_y = Effective Length factor for buckling about y-axis
 L_y = Unbraced length for bending about y-axis
 P_{no} = Nominal axial strength determined in accordance with Section 553.4, with $F_n = F_y$
 C_{mx}, C_{my} = Coefficients whose values are determined in accordance with (a), (b), or (c) as follows:

- a. For compression members in frames subject to joint translation (sideways)

$$C_m = 0.85$$

- b. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

$$C_m = 0.6 - 0.4(M_1/M_2) \quad (553.5-22)$$

where

M_1/M_2 = Ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending.

M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

- c. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m is to be determined by rational analysis. However, in lieu of such analysis, the following values are permitted to be used:

- (i) For members whose ends are restrained,

$$C_m = 0.85, \text{ and}$$

- (ii) For members whose ends are unrestrained,

$$C_m = 1.0.$$

SECTION 554 STRUCTURAL ASSEMBLIES AND SYSTEMS

554.1 Built-Up Sections

554.1.1 Flexural Members Composed of Two Back-to-Back C-Sections

The maximum longitudinal spacing of weld or other connectors, s_{max} , joining two C-sections to form an I-section shall be:

$$s_{max} = \frac{L}{6} \leq \frac{2gT_s}{mq} \quad (554.1-1)$$

where

L = Span of beam

g = Vertical distance between two rows of connections nearest to top and bottom flanges

T_s = Available strength of connection in tension (Section 555)

m = Distance from shear center of one C-section to mid-plane of web

q = Design load on beam for spacing of connectors (See below for methods of determination.)

The load, q , shall be obtained by dividing the concentrated loads or reactions by the length of bearing. For beams designed for a uniformly distributed load, q shall be taken as equal to three times the uniformly distributed load, based on the critical load combinations for ASD, LRFD, and LSD. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, s , the available strength of the welds or connections closest to the load or reaction shall be calculated as follows:

$$T_s = \frac{P_s m}{2g} \quad (554.1-2)$$

where

P_s = Concentrated load [factored load] or reaction based on critical load combinations for ASD, and LRFD.

The allowable maximum spacing of connections, s_{max} , shall depend upon the intensity of the load directly at the connection. Therefore, if uniform spacing of connections is used over the whole length of the beam, it shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods shall be permitted to be adopted:

1. the connection spacing varies along the beam according to the variation of the load intensity, or
2. reinforcing cover plates welded to the flanges at points where concentrated loads occur. The available shear strength of the connections joining these plates to the flanges is then used for T_s , and g is taken as the depth of the beam.

554.1.2 Compression Members Composed of Two Sections in Contact

For compression members composed of two sections in contact, the available axial strength shall be determined in accordance with Section 553.4.1 (a) subject to the following modification. If the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, KL/r is replaced by $(KL/r)_m$ calculated as follows:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \quad (554.1-3)$$

where

- $(KL/r)_o$ = Overall slenderness ratio of entire section about built-up member axis
- a = Intermediate fastener or spot weld spacing
- r_i = Minimum radius of gyration of full unreduced cross-sectional area of an individual shape in a built-up member

See Section 553.4.1(a) for definition of other symbols.

In addition, the fastener strength and spacing shall satisfy the following:

1. The intermediate faster or spot weld spacing, a , is limited such that a/r_i does not exceed one-half the governing slenderness ratio of the built-up member.
2. The ends of a built-up compression member are connected by a weld having a length not less than the maximum width of the member or by connectors spaced longitudinally not more than 4 diameters apart for a distance equal to 1.5 times the maximum width of the member.
3. The intermediate fastener(s) or weld(s) at any longitudinal member tie location are capable of transmitting a force in any direction of 2.5 percent of the nominal axial strength of the built-up member.

554.1.3 Spacing of Connections in Cover Plated Sections

The spacing, s , in the line of stress, of welds, rivets, or bolts connecting a cover plate, sheet, or a non-integral stiffener in compression to another element shall not exceed (a), (b), and (c) as follows:

- a. that which is required to transmit the shear between the connected parts on the basis of the available strength [factored resistance] per connection specified elsewhere herein;

$$b. \quad 1.16t \sqrt{\frac{E}{f_c}}$$

where

- t = Thickness of the cover plate or sheet
- f_c = Compressive stress at nominal load in the cover plate or sheet

- c. three times the flat width, w , of the narrowest unstiffened compression element tributary to the connections, but need not be less than $1.11t \sqrt{\frac{E}{F_y}}$

$$\text{if } w/t < 0.50 \sqrt{\frac{E}{F_y}} \text{ or } 1.33t \sqrt{\frac{E}{F_y}}$$

if $w/t \geq 0.50 \sqrt{\frac{E}{F_y}}$, unless closer spacing is required by (a) or (b) above.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds, plus 12 mm. In all other cases, the spacing shall be taken as the center-to-center distance between connections.

Exception:

The requirements of this section do not apply to cover sheets that act only as sheathing material and are not considered load-carrying elements.

554.2 Mixed Systems

The design of members in mixed systems using cold-formed steel components in conjunction with other materials shall conform to this Specification and the applicable specification of the other material.

554.3 Lateral and Stability Bracing

Braces shall be designed to restrain lateral bending or twisting of a loaded beam or column, and to avoid local crippling at the points of attachment.

554.3.1 Symmetrical Beams and Columns

Braces and bracing systems, including connections, shall be designed considering strength and stiffness requirements.

554.3.2 C-Section and Z-Section Beams

The following provisions for bracing to restrain twisting of C-sections and Z-sections used as beams loaded in the plane of the web shall apply only when neither flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange. When only the top flange is so connected, see Section 554.6.3.1.

Where both flanges are so connected, no further bracing is required.

554.3.2.1 Neither Flange Connected to Sheathing that Contributes to the Strength and Stability of the C- or Z- section

Each intermediate brace at the top and bottom flanges of C- or Z-section members shall be designed with resistance of P_{L1} and P_{L2} , where P_{L1} is the brace force required on the flange in the quadrant with both x and y axes positive, and P_{L2} is the brace force on the other flange. The x-axis shall be designated as the centroidal axis parallel to the web. The x and y coordinates shall be oriented such that one of the flanges is located in the quadrant with both positive x and y axes. See Figure 554.4-1 for illustrations of coordinate systems and positive force directions.

1. For uniform loads

$$P_{L1} = 1.5\{W_y K' - (W_x/2) + (M_z/d)\} \quad (554.3-1)$$

$$P_{L2} = 1.5\{W_y K' - (W_x/2) - (M_z/d)\} \quad (554.3-2)$$

When the uniform load, W , acts through the plane of the web, i.e., $W_y = W$:

$$P_{L1} = -P_{L2} = 1.5(m/d)W \quad \text{for C section} \quad (554.3-3)$$

$$P_{L1} = P_{L2} = 1.5(I_{xy}/2I_x)W \quad \text{for Z section} \quad (554.3-4)$$

where

W_x, W_y = Components of design factored load W parallel to the x- and y- axis, respectively. W_x and W_y are positive if pointing to the positive x- and y- direction, respectively

where

W = Design load (applied load determined in accordance with the most critical load combinations for ASD or LRFD, whichever is applicable) within a distance of $0.5a$ each side of the brace.

where

a = Longitudinal distance between centerline of braces
 K' = 0 for C-sections
 $= I_{xy}/(2I_x)$ for Z-sections

where

I_{xy} = Product of inertia of full unreduced section
 I_x = Moment of inertia of full unreduced section about x-axis
 M_z = $-W_{xesy} + W_{yesx}$, torsional moment of W about shear center

where

e_{sx}, e_{sy} = Eccentricities of load components measured from the shear center and in the x- and y- directions, respectively
 d = Depth of section
 m = Distance from shear center to mid-plane of web of C-section

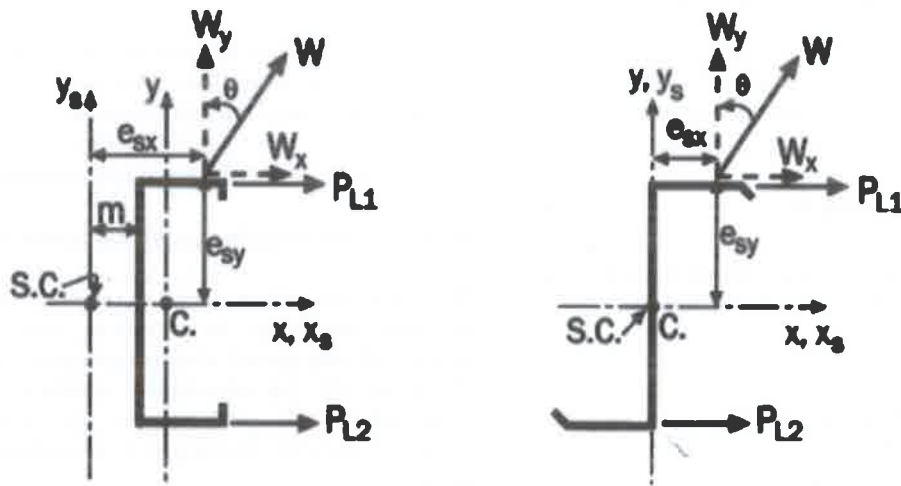


Figure 554.4-1 Coordinate Systems and Positive Force Directions

2. For concentrated loads

$$P_{L1} = P_y K' - (P_x/2) + (M_z/d) \quad (554.3-5)$$

$$P_{L2} = P_y K' - (P_x/2) - (M_z/d) \quad (554.3-6)$$

When a design load [factored load] acts through the plane of the web, i.e., $P_y = P$:

$$P_{L1} = -P_{L2} = (m/d)P \quad \text{for C section} \quad (554.3-7)$$

$$P_{L1} = P_{L2} = (I_{xy}/2I_x)P \quad \text{for Z section} \quad (554.3-8)$$

where

P_x, P_y = Components of design load P parallel to the x - and y - axis, respectively. P_x and P_y are positive if pointing to the positive x - and y -direction, respectively

M_z = $-P_{xesy} + P_{yesx}$, torsional moment of P about shear center

P = Design concentrated load within a distance of $0.3a$ on each side of the brace, plus $1.4(1 - l/a)$ times each design concentrated load located further than $0.3a$ but not farther than $1.0a$ from the brace. The design concentrated load is the applied load factored load determined in accordance with the most critical load combinations for ASD, LRFD, LSD, whichever is applicable

where

l = Distance from concentrated load to the brace

See Section 554.3.2.1(a) for definitions of other variables.

The bracing force, P_{L1} or P_{L2} , is positive where restraint is required to prevent the movement of the corresponding flange in the negative x -direction.

Where braces are provided, they shall be attached in such a manner to effectively restrain the section against lateral deflection of both flanges at the ends and at any intermediate brace points.

When all loads and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the section against torsional rotation and lateral displacement, no additional braces shall be required except those required for strength in accordance with Section 553.3.1b.1.

554.3.3 Bracing of Axially Loaded Compression Members

The required brace strength to restrain lateral translation at a brace point for an individual compression member shall be calculated as follows:

$$P_{br,1} = 0.01 P_n \quad (554.3-9)$$

The required brace stiffness to restrain lateral translation at a brace point for an individual compression member shall be calculated as follows:

$$\beta_{br,1} = \frac{2 \left(4 - \left(\frac{2}{n} \right) \right) P_n}{L_b} \quad (554.3-10)$$

where

- $P_{br,1}$ = Required nominal brace strength resistance for a single compression member
 P_n = Nominal axial compression strength of a single compression member
 $\beta_{br,1}$ = Required brace stiffness for a single compression member
 n = Number of equally spaced intermediate brace locations
 i = Distance between braces on one compression member

554.4 Cold-Formed Steel Light-Frame Construction

The design and installation of structural members and non-structural members utilized in cold-formed steel repetitive framing applications where the specified minimum base steel thickness is between 0.455 mm and 2.997 mm shall be in accordance with the AISI S200 and the following, as applicable:

1. Headers, including box and back-to-back headers, and double and single L-headers, shall be designed in accordance with AISI S212 or solely in accordance with this Specification.
2. Trusses shall be designed in accordance with AISI S214.
3. Wall studs shall be designed in accordance with AISI S211, or solely in accordance with this Specification either on the basis of an all-steel system in accordance with Section 554.4.1 or on the basis of sheathing braced design in accordance with an appropriate theory, tests, or rational engineering analysis. Both solid and perforated webs shall be permitted. Both ends of the stud shall be connected to restrain rotation about the longitudinal stud axis and horizontal displacement

perpendicular to the stud axis.

4. Framing for floor and roof systems in buildings shall be designed in accordance with AISI S210 or solely in accordance with this Specification.

See Section 553-3 for additional requirements.

554.4.1 All-Steel Design of Wall Stud Assemblies

Wall stud assemblies using an all-steel design shall be designed neglecting the structural contribution of the attached sheathings and shall comply with the requirements of Section 553. For compression members with circular or non-circular web perforations, the effective section properties shall be determined in accordance with Section 552.2.2.

554.5 Floor, Roof, or Wall Steel Diaphragm Construction

The in-plane diaphragm nominal shear strength, S_n , shall be established by calculation or test. The safety factors and resistance factors for diaphragms given in Table 554.5-1 shall apply to both methods. If the nominal shear strength is only established by test without defining all limit state thresholds, the safety factors and resistance factors shall be limited by the values given in Table 554-1 for connection types and connection-related failure modes. The more severe factored limit state shall control the design. Where fastener combinations are used within a diaphragm system, the more severe factors shall be used.

Ω_d = As specified in Table 554-1 (ASD)

ϕ_d = As specified in Table 554-1 (LRFD)

Table 554-1 Safety Factors and Resistance Factors for Diaphragms

Load Type or Combinations Including	Connection Type	Limit State			
		Connection		Panel Buckling*	
		Ω_d (ASD)	ϕ_d (LRFD)	Ω_d (ASD)	ϕ_d (LRFD)
Earthquake	Welds	3.00	0.55	2.00	0.80
	Screws	2.50	0.65		
Wind	Welds	2.35	0.70		
	Screws	2.35	0.70		
All Others	Welds	2.65	0.60		
	Screws	2.5	0.65		

Note:

*Panel buckling is out-of-plane and not local buckling at fasteners.

For mechanical fasteners other than screws:

$$M_n = RS_e F_y \quad (554.6-1)$$

1. Ω_d shall not be less than the Table 554-1 values for screws, and
2. ϕ_d shall not be greater than the Table 554-1 values for screws.

In addition, the value of Ω_d and ϕ_d using mechanical fasteners other than screws shall be limited by the Ω and ϕ values established through calibration of the individual fastener shear strength, unless sufficient data exist to establish a diaphragm system effect in accordance with Section 556.1.1. Fastener shear strength calibration shall include the diaphragm material type. Calibration of individual fastener shear strengths shall be in accordance with Section 556.1.1. The test assembly shall be such that the tested failure mode is representative of the design. The impact of the thickness of the supporting material on the failure mode shall be considered.

554.6 Metal Roof and Wall System

The provisions of Section 554.6.1 through 554.6.3 shall apply to metal roof and wall systems that include cold-formed steel purlins, girts, through-fastened wall/roof and wall panels, or standing seam roof panels, as applicable.

554.6.1 Purlins, Girts and Other Members

554.6.1.1 Flexural Members Having One Flange Through-Fastened to Deck or Sheathing

This section shall not apply to a continuous beam for the region between inflection points adjacent to a support or to a cantilever beam.

The nominal flexural strength, M_n , of a C- or Z-section loaded in a plane parallel to the web, with the tension flange attached to deck or sheathing and with the compression flange laterally unbraced, shall be calculated in accordance with Eq. 554.6-1. The safety factor and resistance factors given in this section shall be used to determine the allowable flexural strength or design flexural strength in accordance with the applicable design method in Section 551.4, 551.5.

$$\Omega_b = 1.67 \text{ (ASD)} \quad \phi_b = 0.90 \text{ (LRFD)}$$

where

R is obtained from Table 554.6.1.1-1 for simple span C- or Z-sections, and

$$R = \begin{aligned} &= 0.60 \text{ for continuous span C-sections} \\ &= 0.70 \text{ for continuous span Z-sections} \end{aligned}$$

S_e and F_y = Values as defined in Section 553.3.1.1

The reduction factor, R , shall be limited to roof and wall systems meeting the following conditions:

1. Member depth ≤ 295 mm,
2. Member flanges with edge stiffeners
3. $60 \leq \text{depth / thickness} \leq 170$
4. $2.8 \leq \text{depth / flange width} \leq 4.5$
5. $16 \leq \text{flat width / thickness of flange} \leq 43$,
6. For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) is not less than 1.5d,
7. Member span length is not greater than 10 m
8. Both flanges are prevented from moving laterally at the supports,
9. Roof or wall panels are steel sheets with 340 MPa minimum yield stress, and a minimum of 0.45 mm base metal thickness, having a minimum rib depth of 30 mm, spaced a maximum of 300 mm on centers and attached in a manner effectively inhibit relative movement between the panel and purlin flange,
10. Insulation is glass fiber blanket 0 to 150 mm thick compressed between the member and panel in a manner consistent with the fastener being used,
11. Fastener type is, at minimum, No.12 self-drilling or self-tapping sheet metal screws or 5 mm rivets, having washers 12 mm diameter,
12. Fasteners is not standoff type screws,
13. Fasteners are spaced not greater than 300 mm on centers and placed near the center of the beam flange,

and adjacent to the panel high rib, and

14. The design yield stress of the member does not exceed 410 MPa.

If variables fall outside any of the above stated limits, the user shall perform full-scale tests in accordance with Section 556.1 of this Specification or apply a rational engineering analysis procedure. For continuous purlin systems in which adjacent bay span lengths vary by more than 20 percent, the R values for the adjacent bays shall be taken from Table 554-2. The user shall be permitted to perform tests in accordance with Section 556.1 as an alternate to the procedure described in this section.

Table 554-2 Simple Span C or Z Section R Values

Depth range (mm)	Profile	R
$d \leq 165$	C or Z	0.70
$165 < d \leq 215$	C or Z	0.65
$215 < d \leq 295$	Z	0.50
$215 < d \leq 295$	C	0.40

For simple span members, R shall be reduced for the effects of compressed insulation between the sheeting and the member. The reduction shall be calculated by multiplying R from Table 554-2 by the following correction factor, r :

$$r = 1.00 - 0.0004t_i \text{ when } t_i \text{ is in millimeters} \quad (554.6.1.1-3)$$

where

t_i = Thickness of uncompressed glass fiber blanket insulation

554.6.1.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

See Section 554.6.1b of Section 553-3 or B for the provisions of this section.

554.6.1.3 Compression Members Having One Flange Through-Fastened to Deck or Sheathing

These provisions shall apply to C- or Z-sections concentrically loaded along their longitudinal axis, with only one flange attached to deck or sheathing with through fasteners.

The nominal axial strength of simple span or continuous C- or Z-sections shall be calculated in accordance with (a) and (b).

1. The weak axis nominal strength shall be calculated in accordance with Eq. 554.6-2. The safety factor and resistance factors given in this section shall be used to determine the allowable axial strength or design axial strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$P_n = C_1 C_2 C_3 A E / 29500 \quad (554.6-2)$$

$$\Omega = 1.80 \text{ (ASD)} \quad \phi = 0.85 \text{ (LRFD)}$$

where

$$C_1 = (0.79x + 0.54) \quad (554.6-3)$$

$$C_2 = (1.17\alpha t + 0.93) \quad (554.6-4)$$

$$C_3 = \alpha(2.5b - 1.63d) + 22.8 \quad (554.6-5)$$

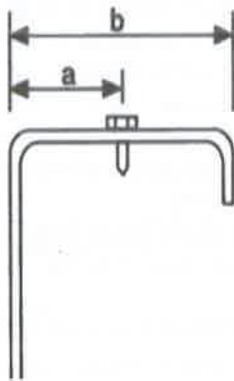
where

- x = For Z-sections, the fastener distance from the outside web edge divided by the flange width, as shown in Figure 554.6.1.3
- = For C-sections, the flange width minus the fastener distance from the outside web edge divided by the flange width, as shown in Figure 554.6.1.3.
- α = Coefficient for conversion of units
- = 0.0394 when t , b , and d are in mm
- t = C- or Z-section thickness
- b = C- or Z-section flange width
- d = C- or Z-section depth
- A = Full unreduced cross-sectional area of C- or Z-section
- E = Modulus of elasticity of steel
- = 203,000 MPa for SI units

Eq. 554.6.1.3-1 shall be limited to roof and wall system meeting the following conditions:

- a. $t \leq 3$ mm,
- b. $150 \text{ mm} \leq d \leq 300 \text{ mm}$,
- c. Flanges are edge stiffened compression elements,
- d. $70 \leq d/t \leq 170$,
- e. $2.8 \leq d/b \leq 5$,
- f. $16 \leq \text{flange flat width}/t \leq 50$

- g. Both flanges are prevented from moving laterally at the supports,
 - h. Steel roof or steel wall panels with fasteners spaced 300 mm on center or less and having a minimum rotational lateral stiffness of 10,300 N/m (fastener at mid-flange width for stiffness determination) determined in accordance with AISI S901,
 - i. C- and Z-sections having a minimum yield stress of 230 MPa, and
 - j. Span length not exceeding 10 m.
2. The strong axis available strength shall be determined in accordance with Sections 553.4.1 and 553.4.1.1.

Figure 554.6.1.3 Definition of x

For Z-section, $x = a/b$ (554.6-6)

For C-section, $x = (b - a)/b$ (554.6-7)

554.6.2 Standing Seam Roof Panel Systems

554.6.2a Strength of Standing Seam Roof Panel Systems

Under gravity loading, the nominal strength of standing seam roof panels shall be determined in accordance with Section 552 and 553 of this Specification or shall be tested in accordance with AISI S906. Under uplift loading, the nominal strength of standing seam roof panel systems shall be determined in accordance with AISI S906. Tests shall be performed in accordance with AISI S906 with the following exceptions:

1. The Uplift Pressure Test Procedure for Class 1 Panel roofs in FM 4471 shall be permitted.

2. Existing tests conducted in accordance with CEGS 07416 uplift test procedure prior to the adoption of these provisions shall be permitted.

The open-open end configuration, although not prescribed by the ASTM E1592 test procedure, shall be permitted provided the tested end conditions represent the installed condition, and the test follows the requirements given in AISI S906. All test results shall be evaluated in accordance with this section.

For load combinations that include wind uplift, additional provisions are provided Section 554.6.2.1a of Section 553-3.

When the number of physical tests assemblies is 3 or more, safety factors and resistance factors shall be determined in accordance with the procedures of Section 554.1.1 (b) with the following definitions for the variables:

- β_o = Target reliability index
 = 2.0 for panel flexural limits
 = 2.5 for anchor limits
- F_m = Mean value of the fabrication factor
 = 1.0
- M_m = Mean value of the material factor
 = 1.1
- V_M = Coefficient of variation of the material factor
 = 0.08 for anchor failure mode
 = 0.10 for other failure modes
- V_F = Coefficient of variation of the fabrication factor
 = 0.05
- V_Q = Coefficient of variation of the load effect
 = 0.21
- V_P = Actual calculated coefficient of variation of the test results, without limit
- n = Number of anchors in the test assembly with the same tributary area (for anchor failure) or number of panels with identical spans and loading to the failed span (for non-anchor failures)

The safety factor, Ω , shall not be less than 1.67, and the resistance factor, ϕ , shall not be greater than 0.9 (LRFD)

When the number of physical test assemblies is less than 3, a safety factor, Ω , of 2.0 and a resistance factor, ϕ , of 0.8 (LRFD) shall be used.

554.6.3 Roof System Bracing and Anchorage

554.6.3.1 Anchorage of Bracing for Purlin Roof Systems Under Gravity Load with Top Flange Connected to Metal Sheathing

Anchorage, in the form of a device capable of transferring force from the roof diaphragm to a support, shall be provided for roof systems with C-sections or Z-sections, designed in accordance with Sections 553.3.1 and 554.6.1, having through-fastened or standing seam sheathing attached to the top flanges. Each anchorage device shall be designed to resist the force, P_L , determined by Eq. 554.6-8 and shall satisfy the minimum stiffness requirement of Eq. 554.6-14. In addition, purlins shall be restrained laterally by the sheathing so that the maximum top flange lateral displacements between lines of lateral anchorage at nominal loads do not exceed the span length divided by 360.

Anchorage devices shall be located in each purlin bay and shall connect to the purlin at or near the purlin top flange. If anchorage devices are not directly connected to all purlin lines of each purlin bay, provision shall be made to transmit the forces from other purlin lines to the anchorage devices. It shall be demonstrated that the required force, P_L , can be transferred to the anchorage device through the roof sheathing and its fastening system. The lateral stiffness of the anchorage device shall be determined by analysis or testing. This analysis or testing shall account for the flexibility of the purlin web above the attachment of the anchorage device connection.

$$P_{Lj} = \sum_{i=1}^{N_p} \left(P_i \frac{K_{effij}}{K_{totali}} \right) \quad (554.6-8)$$

where

P_{Lj} = Lateral force to be resisted by the j^{th} anchorage device (positive when restraint is required to prevent purlins from translating in the upward roof slope direction)

N_p = Number of purlin lines on roof slope

i = Index for each purlin line ($i = 1, 2, \dots, N_p$)

j = Index for each anchorage device ($j = 1, 2, \dots, N_a$)

where

N_a = Number of anchorage devices along a line of anchorage

P_i = Lateral force introduced into the system at the purlin

$$= (C1)W_{Pi} \left\{ \left[\left(\frac{C2}{1000} \right) \frac{I_{xy}L}{I_x d} + (C3) \frac{(m+0.25b)t}{d^2} \right] \alpha \cos \theta - (C4) \sin \theta \right\}$$

(554.6-9)

where

$C1, C2, C3$, and $C4$ = Coefficients tabulated in Tables 554.6.3.1-1 to 554.6.3.1-3

W_{Pi} = Total required vertical load supported by the i^{th} purlin in a single bay

$$= w_i L \quad (554.6-10)$$

where

w_i = Required distributed gravity load supported by the i^{th} purlin per unit length (determined from the critical load combination for ASD, or LRFD)

I_{xy} = Product of inertia of full unreduced section about centroidal axes parallel and perpendicular to the purlin web ($I_{xy} = 0$ for C-sections)

L = Purlin span length

m = Distance from shear center to mid-plane of web ($m = 0$ for Z-sections)

b = Top flange width of purlin

t = Purlin thickness

I_x = Moment of inertia of full unreduced section about centroidal axis perpendicular to the purlin web

d = depth of purlin

α = +1 for top flange facing in the up-slope direction
= -1 for top flange facing in the down slope direction

θ = Angle between vertical and plane of purlin web

K_{effij} = Effective lateral stiffness of the j^{th} anchorage device with respect to the i^{th} purlin

$$= \left[\frac{1}{K_a} + \frac{d_{pij}}{(C6)LA_p E} \right]^{-1} \quad (554.6-11)$$

where

d_{pij} = Distance along roof slope between the i^{th} purlin line and the j^{th} anchorage device

K_a = Lateral stiffness of the anchorage device

$C6$ = Coefficient tabulated in Tables 554.6.3.1-1 to 554.6.3.1-3

A_p = Gross cross-sectional area of roof panel per unit width

E = Modulus of elasticity of steel

K_{totali} = Effective lateral stiffness of all elements resisting force P_i

$$= \sum_{j=1}^{N_a} (K_{effij}) + K_{sys} \quad (554.6-12)$$

where

$$K_{sys} = \text{Lateral stiffness of the roof system, neglecting anchorage devices}$$

$$= \left(\frac{C5}{1000} \right) (N_p) \frac{ELt^2}{d^2} \quad (554.6-13)$$

For multi-span systems, force P_i , calculated in accordance with Eq. 554.6-9 and coefficients **C1** to **C4** from Tables 554-3 to 554-5 for the "Exterior Frame Line", "End Bay", or "End Bay Exterior Anchor" cases, shall not be taken as less than 80 percent of the force determined using the coefficients **C2** to **C4** for the corresponding "All Other Locations" case.

For systems with multiple spans and anchorage devices at supports (support restraints), where the two adjacent bays have different section properties or span lengths than the following procedures shall be used. The values for P_i in Eq. 554.6-8 and Eq. 554.6-14 to 15 shall be taken as the average of the values found from Eq. 554.6-9 evaluated separately for each of the two bays. The values of K_{sys} and K_{effij} in Eq. 554.6-8 and Eq. 554.6-12 shall be calculated using Eq. 554.6-11 and Eq. 554.6-13, with L , t , and d taken as the average values of the two bays.

For systems with multiple spans and anchorage devices at either 1/3 points or mid-points, where the adjacent bays have different section properties or span lengths than the bay under consideration, the following procedures shall be used to account for the influence of the adjacent bays. The value of K_{sys} in Eq. 554.6-12 shall be calculated using Eq. 554.6-13, with L , t , and d taken as the average of the values from the three bays. The values of K_{effij} shall be calculated using Eq. 554.6-11, with L taken as the span length of the bay under consideration. At an end bay, when computing the average values for P_i or averaging the properties for computing K_{sys} , the averages shall be found by adding the value from the first interior bay and two times the value from the end bay and then dividing the sum by three.

The total effective stiffness at each purlin shall satisfy the following equation:

$$K_{totali} \geq K_{req} \quad (554.6-14)$$

where

$$K_{req} = \Omega \left(20 \left| \sum_{i=1}^{N_p} P_i \right| / d \right) \quad (ASD) \quad (554.6-15)$$

$$K_{req} = \left(\frac{1}{\phi} \right) \left(20 \left| \sum_{i=1}^{N_p} P_i \right| / d \right) \quad (LRFD) \quad (554.6-16)$$

$$\Omega = 2.00 \quad (ASD) \quad \phi = 0.75 \quad (LRFD)$$

In lieu of the Eqs. 554.6-8 through 554.6-13, lateral restraint forces shall be permitted to be determined from alternate analysis. Alternate analysis shall include the first or second order effect and account for the effects of roof slope, torsion resulting from applied loads eccentric to shear center, torsion resulting from the lateral resistance provided by the sheathing, and load applied oblique to the principal axes. Alternate analysis shall also include the effects of the lateral and rotational restraint provided by sheathing attached to the top flange. Stiffness of the anchorage device shall be considered and shall account for flexibility of the purlin web above the attachment of the anchorage device connection.

When lateral restraint forces are determined from rational analysis, the maximum top flange lateral displacement of the purlin between lines of lateral bracing at nominal loads shall not exceed the span length divided by 360. The lateral displacement of the purlin top flange at the line of restraint, Δ_{tf} , shall be calculated at factored load levels for LRFD and nominal load levels for ASD and shall be limited to:

$$\Delta_{tf} \leq \left(\frac{1}{\Omega} \right) \left(\frac{1}{20} \right) \quad (ASD) \quad (554.6-17)$$

$$\Delta_{tf} \leq \phi \left(\frac{d}{20} \right) \quad (LRFD) \quad (554.6-18)$$

554.6.3b Alternate Lateral and Stability Bracing for Purlin Roof System

Torsional bracing that prevents twist about the longitudinal axis of a member in combination with lateral that resist lateral displacement of the top flange at the frame line shall be permitted in lieu of the requirements of Section 554.6.3.1. A torsional brace shall prevent torsional rotation of the cross-section at a discrete location along the span of the member. Connection of braces shall be made at or near both flanges of ordinary open sections, including C- and Z-sections. The effectiveness of torsional braces in preventing torsional rotation at the cross-section and the required strength of lateral restraints at the frame line shall be determined by rational engineering analysis or testing. The lateral displacement of the top flange of the C- or Z-section at the frame line shall be limited to $d/(20\Omega)$ for ASD calculated at nominal load [specified load] levels or $\phi d/20$ for LRFD calculated at factored load levels, where d is the depth of the C- or Z-section member, Ω is the safety

factor for ASD, and ϕ is the resistance factor for LRFD. Lateral displacement between frame lines, calculated at nominal load levels, shall be limited to $L/180$, where L is the span length of the member. For pairs of adjacent purlins that provide bracing against twist to each other, external anchorage of torsional brace forces shall not be required.

where

$$\Omega = 2.00 \text{ (ASD)} \quad \phi = 0.75 \text{ (LRFD)}$$

Table 554-3 Coefficients for One Third Point Restraints

		C1	C2	C3	C4	C5	C6	
Simple Span	Through Fastened (TF)		0.5	7.8	42	0.98	0.39	0.40
Multiple Span	Standing Seam (SS)		0.5	7.3	21	0.73	0.19	0.18
	TF	End Bay Exterior Anchor	0.5	15	17	0.98	0.72	0.043
		End Bay Interior Anchor and 1 st Interior Bay Exterior Anchor	0.5	2.4	50	0.96	0.82	0.20
		All Other Locations	0.5	6.1	41	0.96	0.69	0.12
	SS	End Bay Exterior Anchor	0.5	13	13	0.72	0.59	0.035
		End Bay Interior Anchor and 1 st Interior Bay Exterior Anchor	0.5	0.84	56	0.64	0.20	0.14
		All Other Locations	0.5	3.8	45	0.65	0.10	0.014

Table 554-4 Coefficients for Mid Point Restraints

			C1	C2	C3	C4	C5	C6
Simple Span	Through Fastened (TF)		1.0	7.6	44	0.96	0.75	0.42
Multiple Span	Standing Seam (SS)		1.0	7.5	15	0.62	0.35	0.18
	TF	End Bay	1.0	8.3	47	0.95	3.1	0.33
		First Interior Bay	1.0	3.6	53	0.92	3.9	0.36
		All Other Locations	1.0	5.4	46	0.93	3.1	0.31
	SS	End Bay	1.0	7.9	19	0.54	2.0	0.080
		First Interior Bay	1.0	2.5	41	0.47	2.6	0.13
		All Other Locations	1.0	4.1	31	0.46	2.7	0.15

Table 554-5 Coefficients for Support Restraints

			C1	C2	C3	C4	C5	C6
Simple Span	Through Fastened (TF)		0.5	8.2	33	0.99	0.43	0.17
	Standing Seam (SS)		0.5	8.3	28	0.61	0.29	0.051
Multiple Span	TF	End Exterior Frame Line	0.5	14	6.9	0.94	0.073	0.085
		First Interior Frame Line	1.0	4.2	18	0.99	2.5	0.43
		All Other Location	1.0	6.8	23	0.99	1.8	0.36
	SS	End Exterior Frame Line	0.5	13	11	0.35	2.4	0.25
		First Interior Frame Line	1.0	1.7	69	0.77	1.6	0.13
		All Other Locations	1.0	4.3	55	0.71	1.4	0.17

SECTION 555

CONNECTIONS AND JOINTS

555.1 General Provisions

Connections shall be designed to transmit the required strength acting on the connected members with consideration of eccentricity where applicable.

555.2 Welded Connections

The following design criteria shall apply to welded connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is 5 mm or less. For the design of welded connections in which the thickness of the thinnest connected part is greater than 5 mm, refer to the specifications or standards stipulated in the corresponding Section 555.2(a) of Section 553-3 or 552.

Welds shall follow the requirements of the weld standards also stipulated in Section 555.2(a) of Section 553-3 or 552. For diaphragm applications, Section 555.5 shall apply.

555.2.1 Groove Welds in Butt Joints

The nominal strength, P_n , of a groove weld in a butt joint, welded from one or both sides, shall be determined in accordance with (a) or (b), as applicable. The corresponding safety factor and resistance factors shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

1. For tension or compression normal to the effective area or parallel to the axis of the weld, the nominal strength, P_n , shall be calculated in accordance with Eq. 555.2-1:

$$P_n = L t_e F_y \quad (555.2-1)$$

$$\phi = 0.80 \text{ (LRFD)} \quad \Omega = 1.90 \text{ (ASD)}$$

2. For shear on the effective area, the nominal strength, P_n , shall be the smaller value calculated in accordance with Eq. 555.2-2 and 555.2-3:

$$P_n = L t_e 0.6 F_{xx} \quad (555.2-2)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.70 \text{ (ASD)}$$

$$P_n = L t_e F_y \quad (555.2-2)$$

$$\phi = 0.80 \text{ (LRFD)} \quad \Omega = 1.90 \text{ (ASD)}$$

$$P_n = \frac{L t_e F_y}{\sqrt{3}} \quad (555.2-3)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.70 \text{ (ASD)}$$

where

P_n = Nominal strength [resistance] of groove weld

L = Length of weld

t_e = Effective throat dimension of groove weld

F_y = Yield stress of lowest strength base steel

F_{xx} = Tensile strength of electrode classification

555.2.2 Arc Spot Welds

Arc spot welds, where permitted by this Specification, shall be for welding sheet steel to thicker supporting members or sheet-to-sheet in the flat position. Arc spot welds (puddle welds) shall not be made on steel where the thinnest connected part exceeds 4 mm in thickness, nor through a combination of steel sheets having a total thickness over 4 mm.

Weld washers, as shown in Figures 555-1 and 555-2, shall be used where the thickness of the sheet is less than 0.7 mm. Weld washers shall have a thickness between 1.25 mm and 2.0 mm with a minimum pre-punched hole of 9.50 mm diameter. Sheet-to-sheet welds shall not require weld washers.

Arc spot welds shall be specified by minimum effective diameter of fused area, d_e . The minimum allowable effective diameter shall be 9.5 mm.

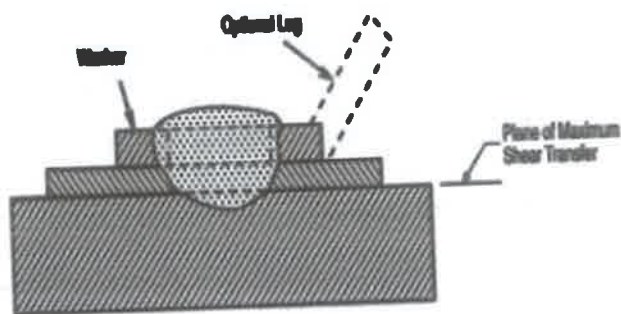


Figure 555-1 Typical Weld Washer

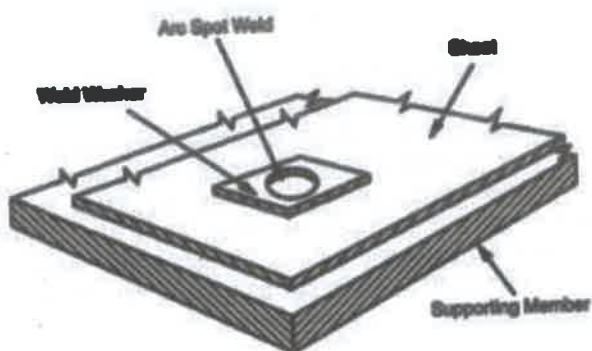


Figure 555-2 Arc Spot Weld Using Washer

555.2.2 Shear

555.2.2.1 Minimum Edge Distance

The distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed shall not be less than the value of e_{min} determined in accordance with Eq. 555.2-4 or Eq. 555.2-5, as applicable. See Figures 555-3 and 555-4 for edge distance of arc welds. The corresponding safety factors and resistance factors shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$e_{min} = \left(\frac{P\Omega}{F_u t} \right) \quad \text{For ASD} \quad (555.2-4)$$

$$e_{min} = \left(\frac{\bar{P}}{\phi F_u t} \right) \quad \text{For LRFD} \quad (555.2-5)$$

when

$$F_u / F_{sy} \geq 1.08$$

$$\Omega = 2.20 \text{ (ASD)} \quad \phi = 0.70 \text{ (LRFD)}$$

$$F_u / F_{sy} < 1.08$$

$$\Omega = 2.55 \text{ (ASD)} \quad \phi = 0.60 \text{ (LRFD)}$$

where

P = Required shear strength (nominal force) transmitted by weld (ASD)

F_u = Tensile strength as determined in accordance with 551.2.1, 551.2.2, or 551.2.3.2

t = Total combined base steel thickness (exclusive of coatings) of sheet(s) involved in shear transfer above plane maximum shear transfer

$\bar{P}$ = Required shear strength transmitted by weld

$$= P_u \text{ (LRFD)}$$

F_{sy} = Yield stress as determined in accordance with Section 551.2.1, 551.2.2, or 551.2.3.2

In addition, the distance from the centerline of any weld to the end or boundary of the connected member shall not be less than $1.5d$. In no case shall the clear distance between welds and the end of member be less than $1.0d$.

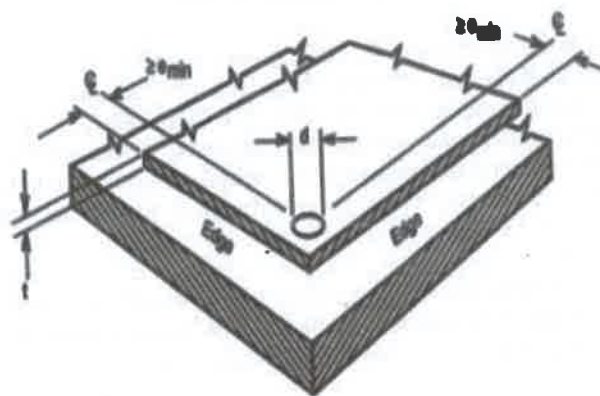


Figure 555-3 Edge Distance for Arc Spot Welds – Single Sheet

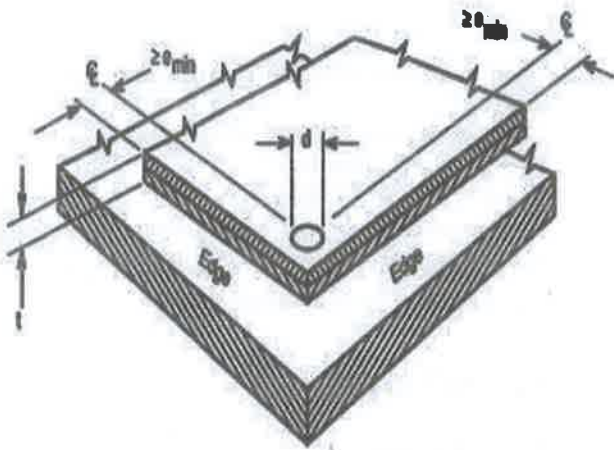


Figure 555-4 Edge Distance for Arc Spot Welds – Double Sheet

555.2.2.2 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member

The nominal shear strength, P_n , of each arc spot weld between the sheet or sheets and a thicker supporting member shall be determined by using the smaller of either (a) or (b). The corresponding safety factor and resistance factors shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section 551.4, or 551.5.

$$1. \quad P_n = \left(\frac{\pi d_e^2}{4} \right) 0.75 F_{xx} \quad (555.2-6)$$

$$\Omega = 2.55 \text{ (ASD)}$$

$$\phi = 0.60 \text{ (LRFD)}$$

$$2. \quad \text{For } (d_a/t) \leq 0.815 \sqrt{\frac{E}{F_u}}$$

$$P_n = 2.20 t d_a F_u \quad (555.2-7)$$

$$\Omega = 2.20 \text{ (ASD)}$$

$$\phi = 0.70 \text{ (LRFD)}$$

$$3. \quad \text{For } 0.815 \sqrt{\frac{E}{F_u}} < (d_a/t) < 1.397 \sqrt{\frac{E}{F_u}}$$

$$P_n = 0.280 \left[1 + 5.59 \sqrt{\frac{E}{F_u}} \frac{d_a}{t} \right] t d_a F_u$$

(555.2-8)

$$\Omega = 2.80 \text{ (ASD)}$$

$$\phi = 0.55 \text{ (LRFD)}$$

$$4. \quad \text{For } (d_a/t) \geq 1.397 \sqrt{\frac{E}{F_u}}$$

$$P_n = 1.40 t d_a F_u \quad (555.2-9)$$

$$\Omega = 3.00 \text{ (ASD)}$$

$$\phi = 0.50 \text{ (LRFD)}$$

where

- P_n = Nominal shear strength of arc spot weld
- d_e = Effective diameter of fused area at plane of maximum shear transfer
- = $0.7d - 1.5t \leq 0.55d$

where

- d = Visible diameter of outer surface of arc spot weld
- t = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer
- F_{xx} = Tensile strength of electrode classification
- d_a = Average diameter of arc spot weld at mid-thickness of t where $d_a = (d - t)$ for single sheet or multiple sheets not more than four lapped sheets over a supporting member. See Figures 555-5 and 555-6 for diameter definitions
- E = Modulus of elasticity of steel
- F_u = Tensile strength as determined in accordance with Section 551.2.1, 551.2.2, or 551.2.3.2

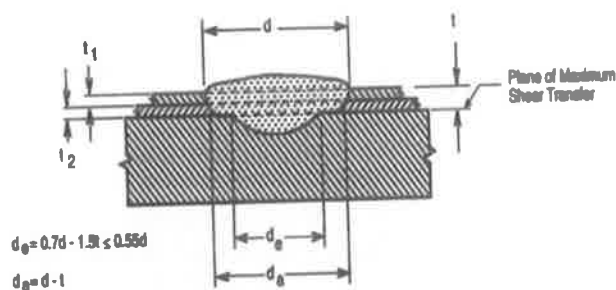


Figure 555-5 Arc Spot Weld – Single Thickness of Sheet

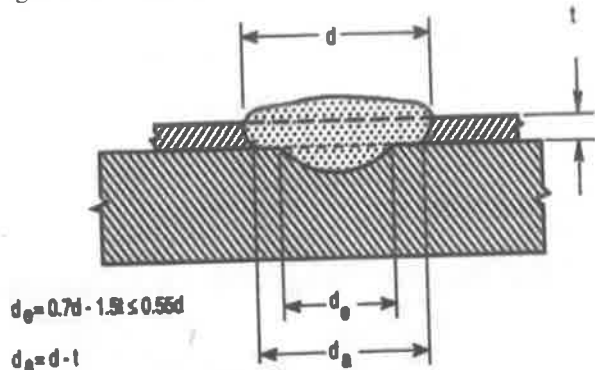


Figure 555-6 Arc Spot Weld – Double Thickness of Sheet

555.2.2.3 Shear Strength [Resistance] for Sheet-to-Sheet Connections

The nominal shear strength [resistance] for each weld between two sheets of equal thickness shall be determined in accordance with Eq. 555.2-10. The safety factor and resistance factors in this section shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$P_n = 1.65 t d_a F_u \quad (555.2-10)$$

$$\Omega = 2.20 \text{ (ASD)} \quad \phi = 0.70 \text{ (LRFD)}$$

where

- P_n = Nominal shear strength [resistance] of sheet-to-sheet connection
- t = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer
- d_a = Average diameter of arc spot weld at mid-thickness of t . See Figure 555-7 for diameter definitions
- = $(d - t)$

where

- d = Visible diameter of the outer surface of arc spot weld
- d_e = Effective diameter of fused area at plane of maximum shear transfer
- = $0.7d - 1.5t \leq 0.55d$ (555.2-11)
- F_u = Tensile strength of sheet as determined in accordance with Section 551.2.1 or 551.2.2

In addition, the following limits shall apply:

$$F_u \leq 407 \text{ Mpa,}$$

$$F_{xx} > F_u, \text{ and}$$

$$0.70 \text{ mm} \leq t \leq 1.60 \text{ mm.}$$

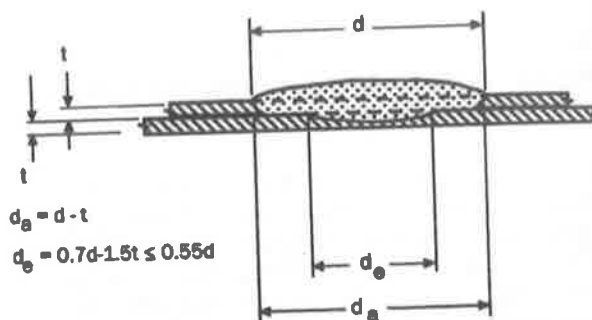


Figure 555.7 Arc Spot Weld – Sheet-to-Sheet

555.2.2.2 Tension

The uplift nominal tensile strength, P_n , of each concentrically loaded arc spot weld connecting sheets and supporting member shall be computed as the smaller of either Eq. 555.2-12 or Eq. 555.2-13 as follows. The safety factor and resistance factors shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, 551.5.

$$P_n = \frac{\pi d_e^2}{4} F_{xx} \quad (555.2-12)$$

$$P_n = 0.8(F_u/F_y) 2 t d_a F_u \quad (555.2-13)$$

For panel and deck applications:

$$\Omega = 2.50 \text{ (ASD)} \quad \phi = 0.60 \text{ (LRFD)}$$

For panel and deck applications:

$$\Omega = 3.00 \text{ (ASD)} \quad \phi = 0.50 \text{ (LRFD)}$$

The following limits shall apply:

1. $t d_a F_u \leq 13.34 \text{ kN}$,
2. $e_{min} \geq d$,
3. $F_{xx} \geq 410 \text{ MPa}$
4. $F_u \leq 565 \text{ MPa}$ (of connecting sheets), and
5. $F_{xx} > F_u$

See Section 555.2.2.1 for definitions of variables.

For eccentrically loaded arc spot welds subjected to an uplift tension load, the nominal tensile strength shall be taken as 50 percent of the above value.

For connections having multiple sheets, the strength shall be determined by using the sum of the sheet thickness as given by Eq. 555.2.2-2.

At the side lap connection within a deck system, the nominal tensile strength of the weld connection shall be 70 percent of the above values.

Where it is shown by measurement that a given weld procedure consistently gives a larger effective diameter, d_e , or average diameter, d_a , as applicable, this larger diameter shall be permitted to be used provided the particular welding procedure used for making those welds is followed.

555.2.3 Arc Seam Welds

Arc seam welds (See Figure E2.3-1) covered by this Specification shall apply on to the following joints:

1. Sheet to thicker supporting member in the flat position, and
2. Sheet to sheet in the horizontal or flat position.

The nominal shear strength, P_n , of arc seam welds shall be determined by using the smaller of either Eq. 555.2-14 or Eq. 555.2-15. The safety factor and resistance factors in this section shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, 551.5.

$$P_n = \left[\frac{\pi d_e^2}{4} + L d_e \right] 0.75 F_{xx} \quad (555.2-14)$$

$$P_n = 2.5t F_u (0.25L + 0.96d_a) \quad (555.2-15)$$

$$\Omega = 2.55 \text{ (ASD)} \quad \phi = 0.60 \text{ (LRFD)}$$

where

$$\begin{aligned} P_n &= \text{Nominal shear strength [resistance] of arc seam weld} \\ d_e &= \text{Effective width of seam weld at fused surfaces} \\ &= 0.7d - 1.5t \end{aligned} \quad (555.2-16)$$

where

$$\begin{aligned} d &= \text{Width of arc seam weld} \\ L &= \text{Length of seam weld not including circular ends} \\ &\quad \text{(For computation purposes, } L \text{ shall not exceed } 3d) \\ d_a &= \text{Average width of seam weld} \\ &= (d - t) \text{ for single or double sheets} \quad (555.2-17) \\ F_u, F_{xx}, \text{ and } t &= \text{Values as defined in Section 555.2.2.1} \end{aligned}$$

The minimum edge distance shall be as determined for the arc spot weld in accordance with Section 555.2.2.1. See Figure 555.9 for details.

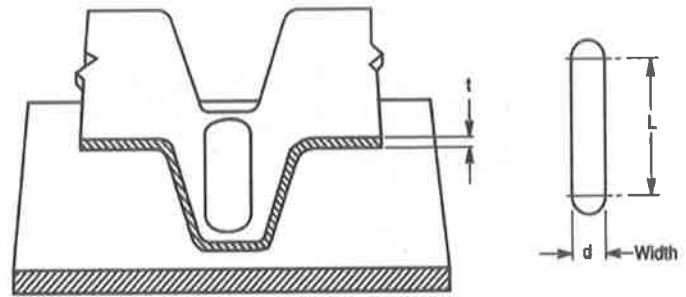


Figure 555-8 Arc Seam Welds – Sheet to Supporting Member in Flat Position

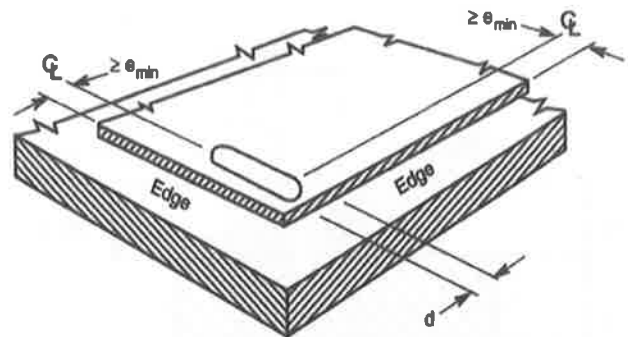


Figure 555-9 Edge Distances for Arc Seam Welds

555.2.4 Fillet Welds

Fillet welds covered by this Specification shall apply to the welding of joints in any position, either sheet to sheet, or sheet to thicker steel member.

The nominal shear strength, P_n , of a fillet weld shall be determined in accordance with this section. The corresponding safety factors and resistance factors given in this section shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

1. For longitudinal loading:

For $L/t < 25$

$$P_n = \left(1 - \frac{0.01L}{t}\right) L t F_u \quad (555.2-18)$$

$$\Omega = 2.55 \text{ (ASD)} \quad \phi = 0.60 \text{ (LRFD)}$$

For $L/t \geq 25$

$$P_n = 0.75 t L F_u \quad (555.2-19)$$

$$\Omega = 3.05 \text{ (ASD)} \quad \phi = 0.50 \text{ (LRFD)}$$

2. For transverse loading:

$$P_n = t L F_u \quad (555.2-20)$$

$$\Omega = 2.35 \text{ (ASD)} \quad \phi = 0.65 \text{ (LRFD)}$$

where

t = Least value of t_1 or t_2 , as shown in Figures 555.10 and 555.11)

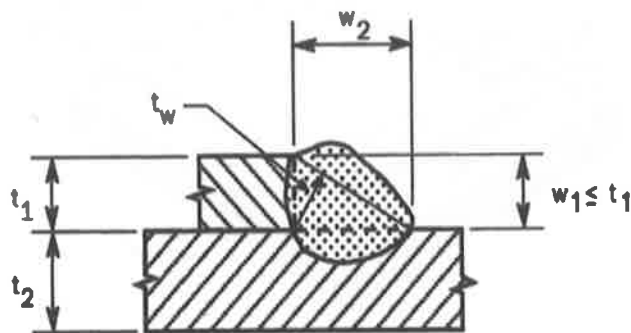


Figure 555-10 Fillet Welds – Lap Joint

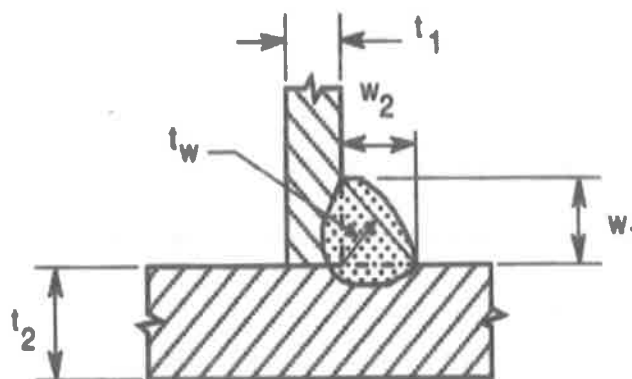


Figure 555-11 Fillet Welds – T Joint

In addition, $t > 2.50$ mm, the nominal strength determined in accordance with (1) and (2) shall not exceed the following value of P_n :

$$P_n = 0.75 t_w L F_{xx} \quad (555.2-21)$$

$$\Omega = 2.55 \text{ (ASD)} \quad \phi = 0.60 \text{ (LRFD)}$$

where

P_n = Nominal strength of fillet weld

L = Length of fillet weld

F_u and F_{xx} = Values as defined in Section 555.2.2.1

t_w = Effective throat

= $0.707w_1$ or $0.707w_2$, whichever is smaller. A larger effective throat is permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w

where

w_1 and w_2 = leg of weld (see Figures 555-10 and 555-11) and $w_1 \leq t_1$ in lap joints

555.2.5 Flare Groove Welds

Flare groove welds covered by this Specification shall apply to welding of joints in any position, either sheet to sheet for flare-V groove welds, sheet to sheet for flare-bevel groove welds or sheet to thicker steel member for flare-bevel groove welds.

The nominal shear strength, P_n , of a flare groove weld shall be determined in accordance with this section. The corresponding safety factors and resistance factors given in this section shall be used to determine the allowable

strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

1. For flare-bevel groove welds, transverse loading (see Figure 555-12)

$$P_n = 0.833 t L F_u \quad (555.2-22)$$

$$\Omega = 2.55 \text{ (ASD)} \quad \phi = 0.60 \text{ (LRFD)}$$

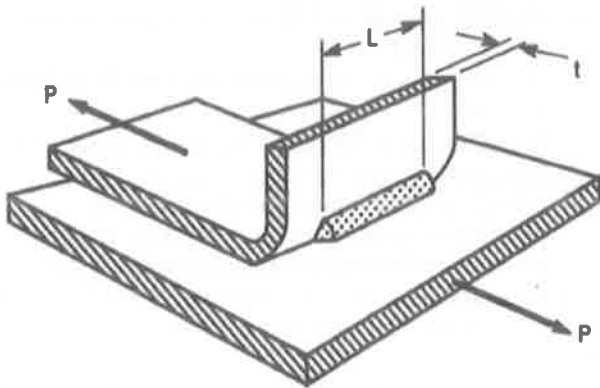


Figure 555-12 Flare-Bevel Groove Weld

2. For flare groove welds, longitudinal loading (see Figures 555-13 through 555-18):
 - a. For $t \leq t_w < 2t$ or if the lip height, h , is less than weld length, L :

$$P_n = 0.75 t L F_u \quad (555.2-23)$$

$$\Omega = 2.80 \text{ (ASD)} \quad \phi = 0.55 \text{ (LRFD)}$$

- b. For $t_w \geq 2t$ with the lip height, h , equal to or greater than weld length, L :

$$P_n = 1.50 t L F_u \quad (555.2-24)$$

$$\Omega = 2.80 \text{ (ASD)} \quad \phi = 0.55 \text{ (LRFD)}$$

In addition, for $t > 2.50$ mm, the nominal strength determined in accordance with (a) and (b) shall not exceed the value of P_n calculated in accordance with Eq. 555.2-25)

$$P_n = 0.75 t_w L F_{xx} \quad (555.2-25)$$

$$\Omega = 2.55 \text{ (ASD)} \quad \phi = 0.60 \text{ (LRFD)}$$

where

P_n = Nominal strength of flare groove weld

t = Thickness of welded member as defined in Figures 555-12 to 555-18

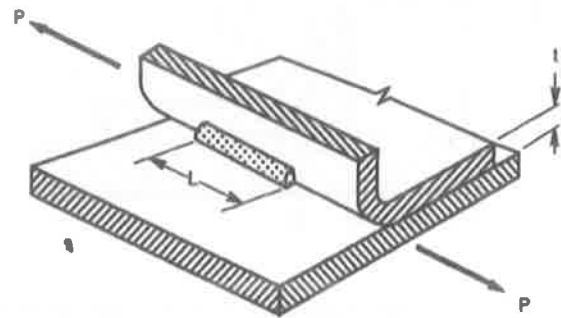


Figure 555-13 Shear in Flare Bevel Groove Weld

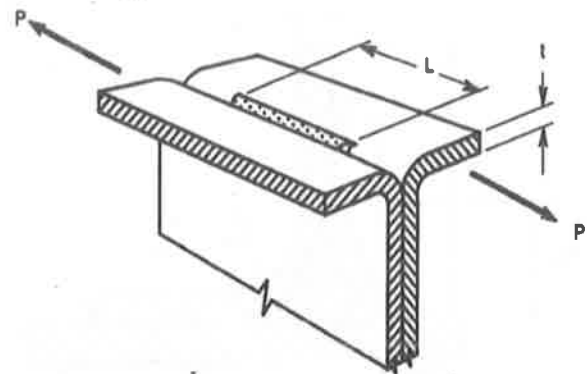


Figure 555-14 Shear in Flare V-Groove Weld

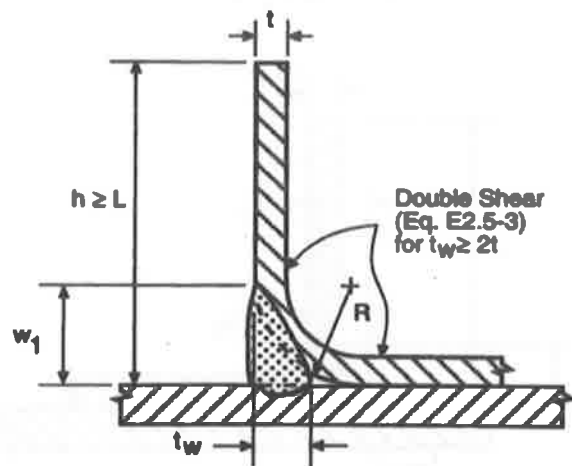


Figure 555-15 Flare Bevel Groove Weld (Filled flush to surface, $w_1 = R$)

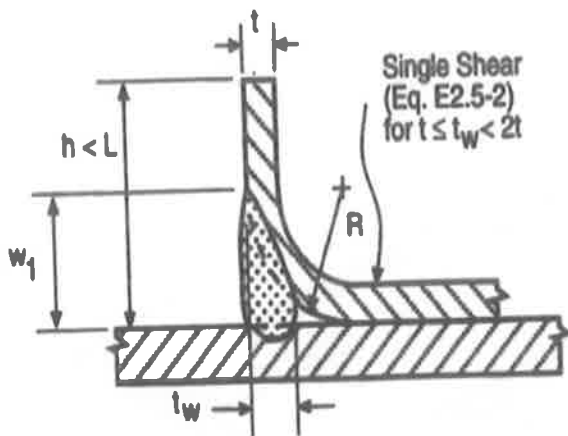


Figure 555-16 Flare Bevel Groove Weld (Filled flush to surface, $w_1 = R$)

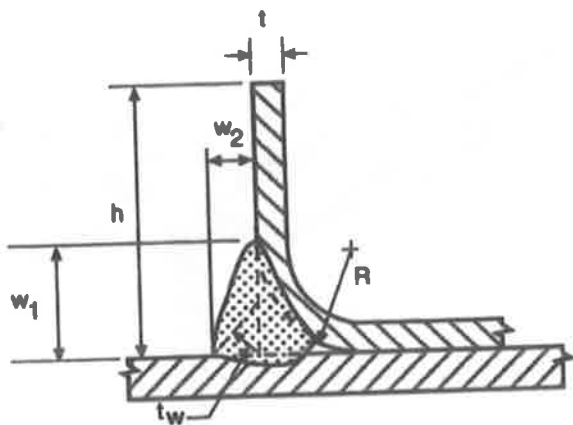


Figure 555-17 Flare Bevel Groove Weld (Not filled flush to surface, $w_1 > R$)

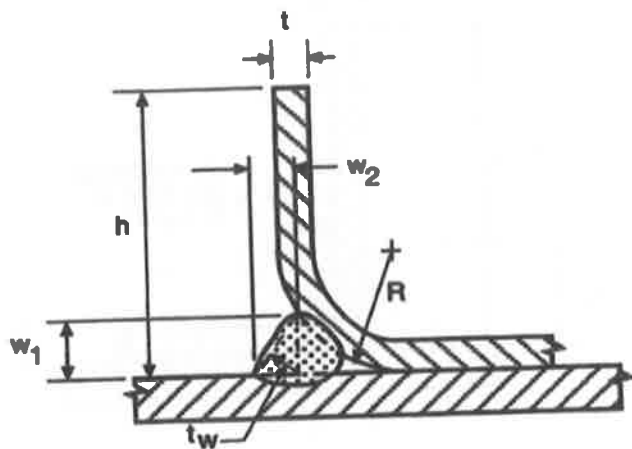


Figure 555-18 Flare Bevel Groove Weld (Not filled flush to surface, $w_1 < R$)

- L = Length of weld
- F_u and F_{xx} = Values as defined in Section 555.2.2.1
- h = Height of lip
- t_w = Effective throat of flare groove weld filled flush to surface (See Figure 555-15 and 555-16):
 - = $(5/16)R$ for flare bevel groove weld
 - = $(1/2)R$ when $R \leq 1/2$ in. (12.77 mm) for flare V-groove weld
 - = $(3/8)R$ when $R > 1/2$ in. (12.77 mm) for flare V-groove weld
- = Effective throat of flare groove weld not filled flush to surface:
 - = $0.707w_1$ or $0.707w_2$, whichever is smaller (see Figures 555-17 and 555-18)
 - = A larger effective throat than those above is permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w

where

- R = Radius of outside bend surface
- w_1 and w_2 = Leg of weld (see Figures 555-17 and 555-18)

555.2.6 Resistance Welds

The nominal shear strength, P_n , of spot welds shall be determined in accordance with this section. The safety factor and resistance factors given in this section shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$\Omega = 2.35 \text{ (ASD)} \quad \phi = 0.65 \text{ (LRFD)}$$

1. When t is in millimeters and P_n is in kN:

For $0.25 \text{ mm} \leq t \leq 3.56 \text{ mm}$

$$P_n = 5.51t^{1.47} \quad (555.2-26)$$

For $3.56 \text{ mm} \leq t \leq 4.57 \text{ mm}$

$$P_n = 7.6t + 8.57 \quad (555.2-27)$$

where

- P_n = Nominal strength [resistance] of resistance weld
- t = Thickness of thinnest outside sheet

555.2.7 Rupture in Net Section of Members other than Flat Sheets (Shear Lag)

The nominal tensile strength of a welded member shall be determined in accordance with Section 553.2. For rupture and/or yielding in the effective net section of the connected part, the nominal tensile strength, P_n , shall be determined in accordance with Eq. 555.2-28. The safety factor and resistance factors given in this section shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$P_n = A_e F_u \quad (555.2-28)$$

$$\Omega = 2.50 \text{ (ASD)} \quad \phi = 0.60 \text{ (LRFD)}$$

where

F_u = Tensile strength of the connected part as determined in accordance with Section 551.2.1 or 551.2.3.2.

A_e = AU , effective net area with U defined as follows:

When the load is transmitted only by transverse welds:

A = Area of directly connected elements

U = 1.0

When the load is transmitted only by longitudinal welds or by longitudinal welds in combination with transverse welds:

A = Gross area of member, A_g

U = 1.0 for members when load is transmitted directly to all of the cross-sectional elements

Otherwise the reduction coefficient U shall be determined in accordance with (a) or (b):

For angle members

$$U = 1.0 - 1.20\bar{x}/L < 0.9 \quad (555.2-29)$$

but $U \geq 0.4$

For channel members

$$U = 1.0 - 0.36\bar{x}/L < 0.9 \quad (555.2-30)$$

but $U \geq 0.5$

where

$\bar{x}$ = Distance from shear plane to centroid of cross-section

L = Length of longitudinal weld

555.3 Bolted Connection

The following design criteria and the requirements stipulated in Section 555.3(a) of Section C-1 and C-2 shall apply to bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than 5 mm. For bolted connection in which the thickness of the thinnest connected part is equal to or greater than 5 mm, the specifications and standards stipulated in Section 555.3(a) of Section 553-3 or 552 shall apply.

Bolts, nuts, and washers conforming to one of the following ASTM specification shall be approved for use under this Specification:

ASTM A194/ A194M, Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service

ASTM A307 (Type A), Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength

ASTM A325, Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

ASTM A325M, High Strength Bolts for Structural Steel Joints [Metric]

ASTM A354 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than 12 mm.)

ASTM A449, Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than 12 mm)

ASTM A490, Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength

ASTM a490M, High Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]

ASTM A563, Carbon and Alloy Steel Nuts

ASTM A563M, Carbon and Alloy Steel Nuts [Metric]

ASTM F436, Hardened Steel Washers

ASTM F36M, Hardened Steel Washers [Metric]

ASTM F844, Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959, Compressible Washer-type Direct Tension Indicators for Use with Structural Fasteners

ASTM F959M, Compressible Washer-Type Direct Tension

Indicators for Use with Structural Fasteners [Metric]

When other than the above are used, drawings shall indicate clearly the type and size of fasteners to be employed and the nominal strength assumed in design.

Bolts shall be installed and tightened to achieve satisfactory performance of the connections.

555.3.1 Shear, Spacing, and Edge Distance

See Section 555.3.1 of the Section 553-3 or B for the provisions of this section

555.3.2 Rupture in Net Section (Shear Lag)

See Section 555.3.2 of the Section 553-3 or 552 for the provisions of this section.

555.3.3 Bearing

The nominal bearing strength of bolted connections shall be determined in accordance with Sections 555.3.3.1 and 555.3.3.2. For conditions not shown, the available bearing strength of bolted connections shall be determined by tests.

555.3.3.1 Strength without Consideration of Bolt Hole Deformation

When deformation around the bolt holes is not a design consideration, the nominal bearing strength, P_n , of the connected sheet for each loaded bolt shall be determined in accordance with Eq. 555.3-1. The safety factor and resistance factors given in this section shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$P_n = C m_f d t F_u \quad (555.3-1)$$

$$\Omega = 2.50 \text{ (ASD)} \quad \phi = 0.60 \text{ (LRFD)}$$

where

- C = Bearing factor, determined in accordance with Table 555.3-1
- m_f = Modification factor for type of bearing connection, which shall be determined according to Table 555.3-2
- d = Nominal bolt diameter
- t = Uncoated sheet thickness
- F_u = Tensile strength of sheet as defined in Section 551.2.1 or 551.2.2

Table 555.3-1 Bearing Factor, C

Thickness of Connected Part 1, mm	Ratio of Fastener Diameter to Member Thickness, d/t	C
$0.60 \leq t \leq 5.0$	$d/t < 10$	3.0
	$10 \leq d/t \leq 22$	$4 - 0.1(d/t)$
	$d/t > 22$	1.8

Table 555.3-2 Modification Factor, m_f , for Type of Bearing Connection

Type of Bearing Connection	m_f
Single Shear and Outside Sheets of Double Shear Connection with Washers under Both Bolt Head and Nut	1.00
Single Shear and Outside Sheets of Double Shear Connection without Washers under Both Bolt Head and Nut, or with only One Washer	0.75
Inside Sheet of Double Shear Connection with or without Washers	1.33

555.3.3.2 Strength with Consideration of Bolt Hole Deformation

When deformation around a bolt hole is a design consideration, the nominal bearing strength, P_n , shall be calculated in accordance with Eq. 555.3-2. The safety factor and resistance factors given in this section shall be used to determine the available strength factored in accordance with the applicable design method in Section 551.4, 551.5. In addition, the available strength shall not exceed the available strength obtained in accordance with Section 555.3.3.1.

$$P_n = (4.64 \alpha t + 1.53) d t F_u \quad (555.3-2)$$

$$\Omega = 2.22 \text{ (ASD)} \quad \phi = 0.65 \text{ (LRFD)}$$

where

- α = Coefficient for conversion of units
= **0.0394** for SI units (with t in mm)

See Section 555.3.3.1 for definitions of other variables

555.3.4 Shear and Tension in Bolts

See Section 555.3.4 of the Section 553-3 or 552 for provisions provided in this section.

555.4 Screw Connections

All Section 555.4 requirements shall apply to screws with $(2.0 \text{ mm}) \leq d \leq 6.5 \text{ mm}$. The screws shall be thread-forming or thread-cutting, with or without a self-drilling point. Screws shall be installed and tightened in accordance with the manufacturer's recommendations.

The nominal screw connection strengths shall also be limited by Section 553.2.

For diaphragm applications, Section 554.5 shall be used.

Except where otherwise indicated, the following safety factor or resistance factor shall be used to determine the allowable strength or design strength in accordance with the applicable design method in Section 551.4, or 551.5.

$$\Omega = 3.00 \text{ (ASD)} \quad \phi = 0.50 \text{ (LRFD)}$$

Alternatively, design values for a particular application shall be permitted to be based on tests, with the safety factor, Ω , and the resistance factor, ϕ , determined according to Section 556.

The following notation shall apply to Section 555.4:

d	=	Nominal screw diameter
d_h	=	Screw head diameter or hex washer head integral washer diameter
d_w	=	Steel washer diameter
d'_w	=	Effective pull-over resistance diameter
P_{ns}	=	Nominal shear strength per screw
P_{ss}	=	Nominal shear strength of screw as reported by manufacturer or determined by independent laboratory testing
P_{not}	=	Nominal pull-out strength per screw
P_{nov}	=	Nominal pull-over strength per screw
P_{ts}	=	Nominal tension strength of screw as reported by manufacturer or determined by independent laboratory testing
t_1	=	Thickness of member in contact with screw head or washer
t_2	=	Thickness of member not in contact with screw head or washer
t_c	=	Lesser of depth of penetration and thickness t_2
F_{u1}	=	Tensile strength of member in contact with screw head or washer
F_{u2}	=	Tensile strength of member not in contact with screw head or washer

555.4.1 Minimum Spacing

The distance between the centers of fasteners shall not be less than $3d$.

555.4.2 Minimum Edge and End Distances

The distance from the center of a fastener to the edge of any part shall not be less than $1.5d$. If the end distance is parallel to the force on the fastener, the nominal shear strength per screw, P_{ns} , shall be limited by Section 555.4.3.2.

555.4.3 Shear

555.4.3.1 Connection Shear Limited by Tilting and Bearing

The nominal shear strength per screw, P_{ns} , shall be determined in accordance with this section.

- For $t_2/t_1 \leq 1.0$, P_{ns} shall be taken as the smallest of

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} \quad (555.4-1)$$

$$P_{ns} = 2.7t_1 d F_{u1} \quad (555.4-2)$$

$$P_{ns} = 2.7t_2 d F_{u2} \quad (555.4-3)$$

- For $t_2/t_1 \geq 2.5$, P_{ns} shall be taken as the smaller of

$$P_{ns} = 2.7t_1 d F_{u1} \quad (555.4-4)$$

$$P_{ns} = 2.7t_2 d F_{u2} \quad (555.4-5)$$

- For $1.0 < t_2/t_1 < 2.5$, P_{ns} shall be calculated by linear interpolation between the above two cases.

555.4.3.2 Connection Shear Limited by End Distance

See Section 555.4.3.2 of the Section 553-3 or 552 for provisions of this section.

555.4.3.3 Shear in Screws

The nominal shear strength of the screw shall be taken as P_{ss} .

In lieu of the value provided Section 555.4, the safety factor or the resistance factor shall be permitted to be determined in accordance with Section 556.1 and shall be taken as $1.25\Omega \leq 3.0 \text{ (ASD)}$, or $\phi/1.25 \geq 0.5 \text{ (LRFD)}$.

555.4.4 Tension

For screws that carry tension, the head of the screw or washer, if a washer is provided, shall have a diameter d_h or d_w not less than 8 mm. Washers shall be at least 1.3 mm thick.

555.4.4.1 Pull-Out

The nominal pull-out strength, P_{not} , shall be calculated as follows:

$$P_{not} = 0.85t_c d F_{u2} \quad (555.4-6)$$

555.4.4.2 Pull-Over

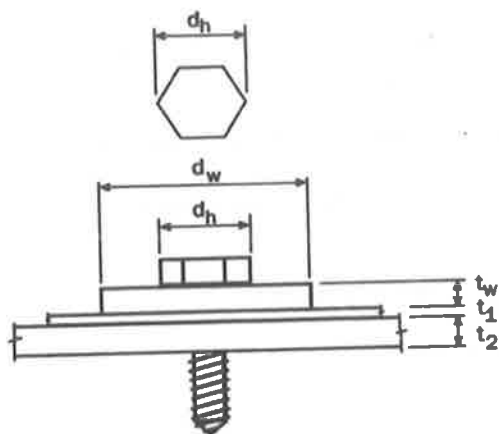
The nominal pull-over strength [resistance], P_{nov} , shall be calculated as follows:

$$P_{nov} = 1.5t_1 d'_w F_{u1} \quad (555.4-7)$$

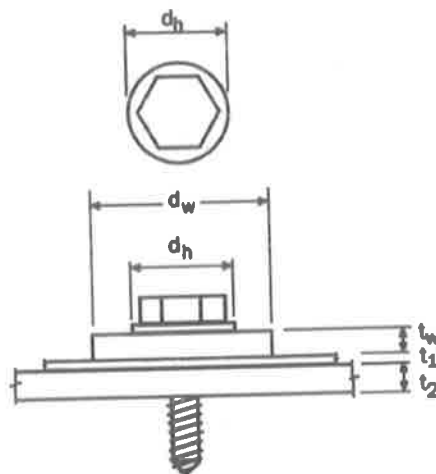
where

d'_w = Effective pull-over diameter determined in accordance with (a), (b), or (c) as follows:

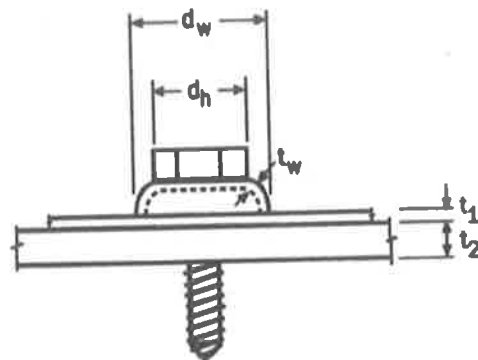
- For a round head, a hex head (Figure 555-19), or hex washer head (Figure 555-19 (2)) screw with an independent and solid steel washer beneath the screw head.



(1) Flat Steel Washer beneath Hex Head Screw Head



(2) Flat Steel Washer beneath Hex Washer Screw Head (HWH has Integral Solid Washer)



(3) Domed Washer (Non-Solid) beneath Screw Head
Figure 555.19 Screw Pull-Over with Washer

$$d'_w = d_h + 2t_w + t_1 \leq d_w \quad (555.4-8)$$

where

- d_h = Screw head diameter or hex washer head integral washer diameter
 t_w = Steel washer thickness
 d_w = Steel washer diameter

- For a round head, a hex head, or hex washer head screw without an independent washer beneath the screw head:

$$d'_w = d_h \text{ but not larger than 12 mm}$$

3. For a domed (non-solid and independent) washer beneath the screw head (Figure 555-19(3)), it is permissible to use d'_w as calculated in Eq. 555.4-8, with d_h , t_w , and t_1 as defined in Figure 555.19(3). In the equation, d'_w cannot exceed 16 mm. Alternatively, pull-over design values for domed washers, including the safety factor, Ω , and the resistance factor, ϕ , shall be permitted to be determined by test in accordance with Section 556.

555.4.4.3 Tension in Screws

The nominal tension strength of the screw shall be taken as P_{ts} .

In lieu of the value provided in Section 555.4, the safety factor or the resistance factor shall be permitted to be determined in accordance with Section 556.1 and shall be taken as $1.25\Omega \leq 3.0$ (ASD), or $\phi/1.25 \geq 0.5$ (LRFD).

555.4.5 Combined Shear and Pull-Over

555.4.5.1 ASD Method

For screw connection subjected to a combination of shear and tension forces, the following requirement shall be met:

$$\frac{Q}{P_{ns}} + 0.71 \frac{T}{P_{nov}} \leq \frac{1.10}{\Omega} \quad (555.4-9)$$

In addition, Q and T shall not exceed the corresponding allowable strength determined by Section 555.4.3 and 555.4.4, respectively.

where

- Q = Required allowable shear strength of connection
 T = Required allowable tension strength of connection
 P_{ns} = Nominal shear strength of connection
 $= 2.7t_1dF_{u1}$ (555.4-10)
 P_{nov} = Nominal pull-over strength of connection
 $= 1.5t_1d_wF_{u1}$ (555.4-11)

where

- d_w = Larger of screw head diameter or washer diameter
 Ω = 2.35

Eq. 555.4-9 shall be valid for connections that meet the following limits:

1. 0.0285 in. (0.724 mm) $\leq t_1 \leq 0.0445$ in. (1.130 mm),
2. No. 12 and No. 14 self-drilling screw with or without washers,
3. $d_w \leq 0.75$ in. (19.1 mm),
4. $F_{u1} \leq 483$ MPa, and
5. $t_2/t_1 \geq 2.5$.

For eccentrically loaded connections that produce a non-uniform pull-over force on the fastener, the nominal pull-over strength shall be taken as 50 percent of P_{nov} .

555.4.5.2 LRFD Method

For screw connections subjected to a combination of shear and tension forces, the following requirements shall be met:

$$\frac{\bar{Q}}{P_{ns}} + 0.71 \frac{\bar{T}}{P_{nov}} \leq 1.10\phi \quad (555.4-12)$$

In addition, Q and T shall not exceed the corresponding design strength [factored resistance] determined in accordance with Section 555.4.3 and 555.4.4, respectively.

where

- Q = Required shear strength of connection
 $= V_u$ for LRFD
 T = Required tension strength of connection
 $= T_u$ for LRFD
 P_{ns} = Nominal shear strength of connection
 $= 2.7t_1dF_{u1}$ (555.4-13)
 P_{nov} = Nominal pull-over strength [resistance] of connection
 $= 1.5t_1d_wF_{u1}$ (555.4-14)

where

- d_w = Larger of screw head diameter or washer diameter
 ϕ = 0.65 LRFD

Eq. 555.4-12 shall be valid connections that meet the following limits:

1. 0.75 mm $\leq t_1 \leq 1.15$ mm,

2. No. 12 and No. 14 self-drilling screw with or without washers,
3. $d_w \leq 19$ mm,
4. $F_{u1} \leq 483$ MPa, and
5. $t_2/t_1 \geq 2.5$.

For eccentrically loaded connections that produce a non-uniform pull-over force on the fastener, the nominal pull-over strength shall be taken as 50 percent of P_{nov} .

555.5 Rupture

See Section 555.5 of Section 553-3 or 552 for the provisions of this section.

555.5 Connecting to Other Materials

555.6.1 Bearing

Provisions shall be made to transfer bearing forces from steel components covered by this Specification to adjacent structural components made of other materials.

555.6.2 Tension

The pull-over shear/ tension forces in the steel sheet around the head of the fastener shall be considered, as well as the pull-out force resulting from axial loads and bending moments transmitted onto the fastener from various adjacent structural components in the assembly.

The nominal tensile strength of the fastener and the nominal embedment strength of the adjacent structural component shall be determined by applicable product code approvals, product specifications, product literature, or combination thereof.

555.6.3 Shear

Provisions shall be made to transfer shearing forces from steel components covered by this Specification to adjacent structural components made of other materials. The required shear and/or bearing strength on the steel components shall not exceed that allowed by this Specification. The available shear strength on the fasteners and other material shall not be exceeded. Embedment requirements shall be met. Provisions shall also be made for shearing forces in combination with other forces.

SECTION 556 TESTS FOR SPECIAL CASES

Tests shall be made by an independent testing laboratory or by a testing laboratory of a manufacturer.

The provisions of Section 556 shall not apply to cold-formed steel diaphragms. Refer to Section 554.5.

556.1 Tests for Determining Structural Performance

556.1.1 Load and Resistance Factor Design and Limit States Design

Any structural performance that is required to be established by tests shall be evaluated in accordance with the following performance procedure:

1. Evaluation of the test results shall be made on the basis of the average value of test data resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test result from the average value obtained from all tests does not exceed ± 15 percent. If such deviation from the average value exceeds 15 percent, more tests of the same kind shall be made until the deviation of any individual test result from the average value obtained from all tests does not exceed ± 15 percent or until at least three additional tests have been made. No test result shall be eliminated unless a rationale for its exclusion is given. The average value of all tests made shall then be regarded as the nominal strength, R_n , for the series of the tests. R_n and the coefficient of variation V_P of the test results shall be determined by statistical analysis.
2. The strength of the tested elements, assemblies, connections, or members shall satisfy Eq. 556.1-1 or Eq. 556.1-2 as applicable.

$$\Sigma \gamma_i Q_i \leq \phi R_n \quad \text{for LRFD} \quad (556.1-1)$$

where

$\Sigma \gamma_i Q_i$ = Required strength [factored loads] based on the most critical load combination determined in accordance with Section 551.5.1.2 for LRFD. γ_i and Q_i are load factors and load effects, respectively

ϕ = Resistance factor

$$= C_\phi (M_m F_m P_m) e^{\beta_o \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}}$$

(556.1-2)

where

- C_ϕ = calibration coefficient
 = 1.52 for LRFD
 = 1.6 for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced
 M_m = Mean value of material factor, M , listed in Table 556-1 for type of component involved
 F_m = Mean value of fabrication factor, F , listed in Table 556-1 for type of component involved
 P_m = Mean value of professional factor, P , for tested component
 = 1.0
 e = Natural logarithmic base
 = 2.718
 β_o = Target reliability index
 = 2.5 for structural members and 3.5 for connections for LRFD
 = 1.5 for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced
 V_M = Coefficient of variation of material factor listed in Table 556-1 for type of component involved
 V_F = Coefficient of variation of fabrication factor listed in Table 556-1 for type of component involved
 C_P = Correction factor
 = $(1 + 1/n)m/(m - 2)$ for $n \geq 4$
 = 5.7 for $n = 3$

where

- n = Number of tests
 m = Degrees of freedom
 = $n - 1$
 V_P = Coefficient of variation of test results, but not less than 6.5 percent
 V_Q = Coefficient of variation of load effect
 = 0.21 for LRFD
 = 0.43 for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced
 R_n = Average result of all test results

The listing in Table 556-1 shall not exclude the use of other documented statistical data if they are established from sufficient results on material properties and fabrication.

For steels not listed in Section 551.2.1, values of M_M and

V_M shall be determined by the statistical analysis for the materials used.

When distortions interfere with the proper functioning of the specimen in actual use, the load effects based on the critical load combination at the occurrence of the acceptable distortion shall also satisfy Eq. 556.1-1(a) or Eq. 556.1-2, as applicable, except that the resistance factor ϕ shall be taken as unity and the load factor for dead load shall be taken as 1.0.

3. The mechanical properties of the steel sheet shall be determined based on representative samples of the material taken from the test specimen or the flat sheet used to form the test specimen. Mechanical properties reported by the steel supplier shall not be used in the evaluation of the test results. If the yield stress of the steel from which the tested sections are formed is larger than the specified value, the test results shall be adjusted down to the specified minimum yield stress of the steel that the manufacturer intends to use. The test results shall not be adjusted upward if the yield stress of the test specimen is less than the minimum specified yield stress. Similar adjustments shall be made on the basis of tensile strength instead of yield stress where tensile strength is the critical factor.

Consideration shall also be given to any variation or differences between the design thickness and the thickness of the specimens used in the tests.

Table 556-1 Statistical Data for the Determination of Resistance Factor

Type of Component	M_M	V_M	F_m	V_f
Transverse Stiffeners	1.10	0.10	1.00	0.05
Shear Stiffeners	1.00	0.06	1.00	0.05
Tension Members	1.10	0.10	1.00	0.05
Flexural members	1.10	0.10	1.00	0.05
• Bending Strength	1.00	0.06	1.00	0.05
• Lateral Torsional Buckling Strength	1.10	0.10	1.00	0.05
• One Flange Through Fastened to Deck or Sheathing	1.10	0.10	1.00	0.05
• Shear Strength	1.10	0.10	1.00	0.05
• Combined Bending and Shear	1.10	0.10	1.00	0.05
• Web Crippling Strength	1.10	0.10	1.00	0.05
• Combined Bending and Web Crippling	1.10	0.10	1.00	0.05
Concentrically Loaded Compression Members	1.10	0.10	1.00	0.05
Combined Axial and Bending	1.05	0.10	1.00	0.05
Cylindrical Tubular Members	1.10	0.10	1.00	0.05
• Bending Strength	1.10	0.10	1.00	0.05
• Axial Compression	1.10	0.10	1.00	0.05
Wall Studs and Wall Studs Assemblies	1.10	0.10	1.00	0.05
• Wall Studs in Compression	1.10	0.10	1.00	0.05
• Wall Studs in Bending	1.10	0.10	1.00	0.05
• Wall Studs with Combined Axial and Bending	1.05	0.10	1.00	0.05
Structural Members not listed above	1.00	0.10	1.00	0.05
Welded Connections				
Arc Spot Welds				
• Shear Strength of Welds	1.10	0.10	1.00	0.10
• Tensile Strength of Weld	1.10	0.10	1.00	0.10
• Plate Failure	1.10	0.08	1.00	0.15
Arc Seam Welds				
• Shear Strength of the Welds	1.10	0.10	1.00	0.10
• Plate Tearing	1.10	0.10	1.00	0.10
Fillet Welds				
• Shear Strength of	1.10	0.10	1.00	0.10

Welds				
• Plate Failure	1.10	0.80	1.00	0.15
Flare Groove Welds				
• Shear Strength of Welds	1.10	0.10	1.00	0.10
• Plate Failure	1.10	0.10	1.00	0.10
Resistance Welds	1.10	0.10	1.00	0.10
Bolted Connections				
• Shear Strength of Bolts	1.10	0.08	1.00	0.05
• Tensile Strength of Bolts	1.10	0.08	1.00	0.05
• Minimum Spacing and Edge Distance	1.10	0.08	1.00	0.05
• Tension Strength on Net Section	1.10	0.08	1.00	0.05
• Bearing Strength	1.10	0.08	1.00	0.05
Screw Connections				
Shear Strength of Screw	1.10	0.10	1.00	0.10
Tensile Strength of Screw	1.10	0.10	1.00	0.10
Minimum spacing and Edge Distance	1.10	0.10	1.00	0.10
Tension Strength on Net Section	1.10	0.10	1.00	0.10
Tilting and Bearing Strength	1.10	0.08	1.00	0.05
Pull-Out	1.10	0.10	1.00	0.10
Pull-Over	1.10	0.10	1.00	0.10
Combined Shear and Pull-Over	1.10	0.10	1.00	0.10
Connections Not Listed Above	1.10	0.10	1.00	0.15

556.1.2 Allowable Strength Design

Where the composition or configuration of elements, assemblies, connections, or details of cold-formed steel structural members are such that calculation of their strength cannot be made in accordance with the provisions of this Specification, their structural performance shall be established from tests and evaluated in accordance with Section 556.1.1, except as modified in this section for allowable strength design.

The allowable strength shall be calculated as follows:

$$R = R_n / \Omega \quad (556.1-3)$$

where

$$\begin{aligned}
 R_n &= \text{average value of all test results} \\
 \Omega &= \text{Safety factor} \\
 &= \frac{1.6}{\phi} \quad (556.1-4)
 \end{aligned}$$

where

ϕ = A value evaluated in accordance with Section 556.1.1

The required strength shall be determined from nominal loads and load combinations as described in Section 551.4.

556.2 Tests for Confirming Structural Performance

For structural members, connections, and assemblies for which the nominal strength is computed in accordance with this Specification or its specific references, confirmatory tests shall be permitted to be made to demonstrate the strength is not less than the nominal strength, R_n , specified in this Specification or its specific references for the type of behavior involved.

556.3 Tests for Determining Mechanical Properties

556.3.1 Full Section

Tests for determination of mechanical properties of full sections to be used in Section 551.7.2 shall be conducted in accordance with this section.

1. Tensile testing procedures shall agree with ASTM A370.
2. Compressive yield stress determinations shall be made by means of compression tests of short specimens of the section. See AISI S902.

The compressive yield stress shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the cross-sectional area or the stress defined by one of the following methods:

- a. For sharp yielding steel, the yield stress is determined by the autographic diagram method or by the total strain under load method.
- b. For gradual yielding steel, the yield stress is determined by the strain under load method or by the 0.2 percent offset method.

When the total strain under load method is used, there shall be evidence that the yield stress so determined agrees within 5 percent with the yield stress that would be determined by the 0.2 percent offset method.

- c. Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield stress shall be determined for the flanges only. In determining such yield stress, each specimen shall consist of one complete flange plus

a portion of the web of such flat width ratio that the value of ρ for the specimen is unity.

- d. For acceptance and control purposes, one full section test shall be made from each master coil.
- e. At the option of the manufacturer, either tension or compression tests shall be permitted to be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield stress of the section when subjected to the kind of stress under which the member is to be used.

556.3.2 Flat Elements of Formed Sections

Tests for determining mechanical properties of flat elements of formed sections and representative mechanical properties of virgin steel to be used in Section 551.7.2 shall be made in accordance with this section.

The yield stress of flats, F_{yf} , shall be established by means of a weighted average of the yield stresses of standard tensile coupons taken longitudinally from the flat portions of a representative cold-formed member. The weighted average shall be the sum of the products of the average yield stress for each flat portion times its cross-sectional area, divided by the total area of flats in the cross-section. Although the exact number of such coupons will depend on the shape of the member, i.e., on the number of flats in the cross-section, at least one tensile coupon shall be taken from the middle of each flat. If the actual virgin yield stress exceeds the specified minimum yield stress, the yield stress of the flats, F_{yf} , shall be adjusted by multiplying the test values by the ratio of the specified minimum yield stress to the actual virgin yield stress.

556.3.3 Virgin Steel

The following provisions shall apply to steel produced to other than the ASTM Specifications listed in Section 551.2.1 when used in sections for which the increased yield stress of the steel after cold forming is computed from the virgin steel properties in accordance with Section 551.7.2. For acceptance and control purposes, at least four tensile specimens shall be taken from each master coil for the establishment of the representative values of the virgin tensile yield stress and tensile strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.

SECTION 557

DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS AND CONNECTIONS FOR CYCLIC LOADING (FATIGUE)

Since the occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design, the evaluation of fatigue resistance shall not be required for wind load applications in buildings. If the live load stress range is less than the threshold stress range, F_{TH} , given in Table 557.1, evaluation of fatigue strength shall also not be required.

This design procedure shall apply to cold-formed steel structural members and connections subject to cyclic loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure (fatigue).

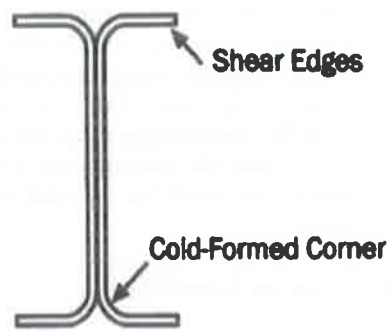
557.1 General

When cyclic loading is a design consideration, the provisions of this chapter shall apply to stresses calculated on the basis of unfactored loads. The maximum permitted tensile stress due to unfactored loads shall be $0.66 F_y$.

Stress range shall be defined as the magnitude of the change in stress due to the application or removal of the unfactored live load. In the case of a stress reversal, the stress range shall be computed as the sum of the absolute values of maximum repeated tensile and compressive stresses or the sum of the absolute values of maximum shearing stresses of opposite direction at the point of probable crack initiation.

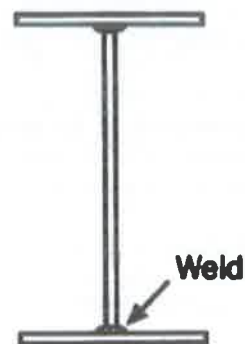
Table 557-1 Fatigue Design parameters for Cold-Formed Steel Structures

Description	Stress Category	Constant C_f	Threshold F_{TH} (Mpa)	Reference Figure
As-received base metal and components with as-rolled surfaces, including sheared edges and cold formed corners	I	3.2×10^{10}	172	557-1
As-received base metal and weld metal in members connected by continuous longitudinal welds	II	1.0×10^{10}	103	557-2
Welded attachments to a plate or a beam, transverse fillet welds, and continuous longitudinal fillet welds less than or equal to 50mm, bolt and screw connections and spot welds	III	3.2×10^9	110	557.1-3 557.1-4
Longitudinal fillet welded attachments greater than 50mm parallel to the direction of the applied stress, and intermittent welds parallel to the direction of the applied force.	IV	1.0×10^9	62	557.1-4



Cold-Formed Steel Channels, Stress Category I

Figure 557-1 Typical Detail for Stress Category I



Welded I Beam, Stress Category II

Figure 557-2 Typical Detail for Stress Category

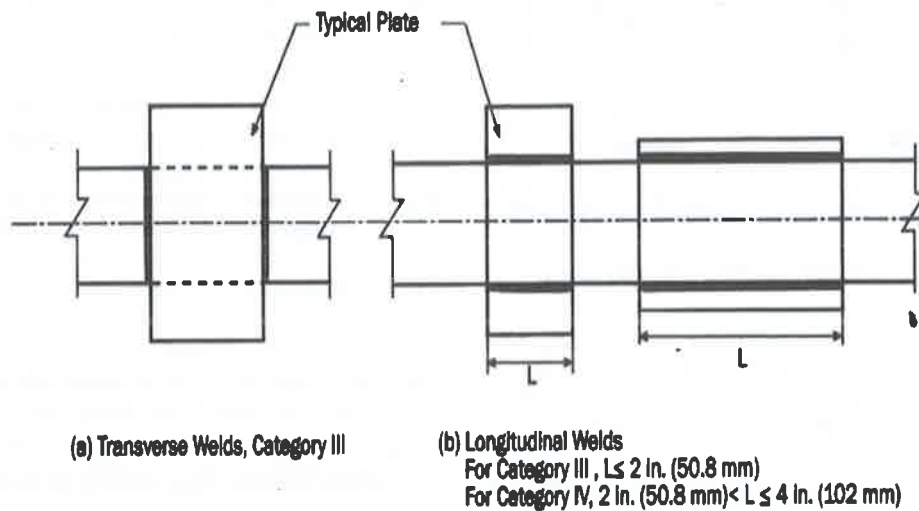


Figure 557-3 Typical Attachments for Stress Categories III and IV

Evaluation of fatigue strength shall not be required if the number of cycles of application of live load is less than 20,000.

The fatigue strength determined by the provisions of this chapter shall be applicable to structures with corrosion protection or subject only to non-aggressive atmospheres.

The fatigue strength determined by the provisions of this chapter shall be applicable only to structures subject to temperatures not exceeding 300°F (149°C).

The contract documents shall either provide complete details including weld sizes, or specify the planned cycle life and the maximum range of moments, shear, and reactions for the connections.

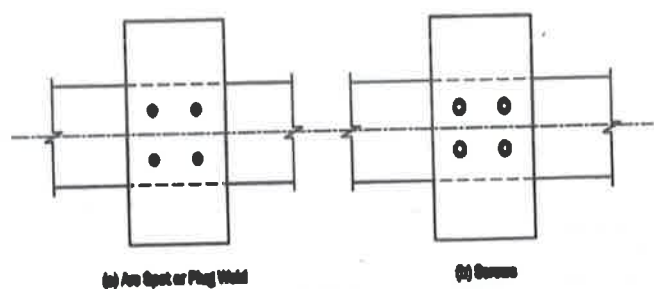


Figure 557-4 Typical Attachments for Stress Category III

557.2 Calculation of Maximum Stresses and Stress Ranges

Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of prying action, if applicable. In the case of axial stress combined with bending, the maximum stresses of each kind shall be those determined for concurrent arrangements of applied load.

For members having symmetric cross-sections, the fasteners and weld shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the stress range.

For axially loaded angle members, where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross-section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range.

557.3 Design Stress Range

The range of stress at service loads [specified] shall not exceed the design stress range computed using Eq. 557-1 for all stress categories as follows:

$$F_{SR} = (\alpha C_f / N)^{0.333} \geq F_{TH} \quad (557.3-1)$$

where

- F_{SR} = Design stress range
- α = Coefficient for conversion of units
329 for SI units
- C_f = Constant from Table 557-1
- N = Number of stress range fluctuations in design life
= Number of stress range fluctuations per day x 365 x years of design life
- F_{TH} = Threshold fatigue stress range, maximum stress range for indefinite design life from Table 557-1

557.4 Bolts and Threaded Parts

For mechanically fastened connections loaded in shear, the maximum range of stress in connected material at service loads shall not exceed the design stress range computed using Eq. 557.3-1. The factor C_f shall be taken as 22×10^8 . The threshold stress, F_{TH} , shall be taken as 48 MPa.

For not-fully-tightened high-strength bolts, and threaded anchor rods with cut, ground, or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial load and moment plus load due to prying action shall not exceed the design stress range computed using Eq. 557.3-1. The factor C_f shall be taken as 3.9×10^8 . The threshold stress, F_{TH} , shall be taken as 48 MPa. The net tensile area shall be calculated by Eqs. 557.4-1.

$$A_t = (\pi/4)[d_b - (0.9382p)]^2 \quad \text{for SI units} \quad (557.4-1)$$

where

- A_t = Net tensile area
- d_b = Nominal diameter (body or shank diameter)

p = Pitch (mm per thread for SI units)

557.5 Special Fabrication Requirements

Backing bars in welded connections that are parallel to the stress field shall be permitted to remain in place, and if used, shall be continuous.

Backing bars that are perpendicular to the stress field, if used, shall be removed and the joint back gouged and welded.

Flame cut edges subject to cyclic stress ranges shall have a surface roughness not to exceed **25 μm** in accordance with ASME B46.1.

Re-entrant corners at cuts, copes, and weld access holes shall form a radius of not less than 10 mm by predrilling or subpunching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal contour to provide a radiused transition, free of notches, with a surface roughness not to exceed not to exceed **25 μm** in accordance with ASME B46.1 or another equivalent approved standards.

For transverse butt joints in regions of high tensile stress, run-off tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Run-off tabs shall be removed and the end of the weld finished flush with the edge of the member.

Exception

Run-off tabs shall not be required for sheet material if the welding procedures used result in smooth, flush edges.

SECTION C-1 DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS USING THE DIRECT STRENGTH METHOD

C-1 Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

C-1.1 General Provisions

C-1.1.1 Applicability

The provisions of this Section shall be permitted to be used to determine the nominal axial (P_n) and flexural (M_n) strengths of cold-formed steel members. Sections C.1.2.1 and C.1.2.2 present a method applicable to all cold-formed steel columns and beams. Those members meeting the geometric and material limitations of Section C.1.1.1.1 for columns and Section C.1.1.1.2 for beams have been prequalified for use, and the calibrated safety factor, Ω , and resistance factor, ϕ , given in C.1.2.1 and C.1.2.2 shall be permitted to apply. The use of the provisions of Section C.1.2.1 and C.1.2.2 for other columns and beams shall be permitted, but the standard Ω and ϕ factors for rational engineering analysis (Section A1.2 (b) of the main Specification) apply. The main American Specification for the Design of Cold-Formed Steel Structural Members.

Currently, the Direct Strength Method provides no explicit provisions for members in tension, shear, combined bending and shear, web crippling, combined bending and web crippling, or combined axial load and bending (beam-columns). Further, no provisions are given for structural assemblies or connections and joints. As detailed in main Specification, Section 551.1.2, the provisions of the main Specification, when applicable, shall be used for all cases listed above.

It shall be permitted to substitute the nominal strength, resistance factors, and safety factors from this Appendix for the corresponding values in Sections 553.3.1, 553.4.1.1, 553.4.1.2, 553.4.1.3, 553.4.1.4, 554.6.1.1, and 554.6.1.2 of the main Specification.

For members of situations to which the main Specification is not applicable, the Direct Strength Method of this Appendix shall be permitted to be used, as applicable. The usage of the engineering analysis procedure, as detailed in Section 551.1.2 (b) of the main Specification:

1. Applicable provisions of the main Specification shall be followed when they exist, and
2. Increased safety factors, Ω , and reduced resistance factors, ϕ , shall be employed for strength when rational engineering analysis is conducted.

C.1.1.1.1 Pre-qualified Columns

Unperforated columns that fall within the geometric and material limitations given in Table C-1 shall be permitted to be designed using the safety factor, Ω , and resistance factor, ϕ , defined in Section C.1.2.1.

C.1.1.1.2 Pre-qualified Beams

Unperforated beams that fall within the geometric and material limitations given in Table C-2 shall be permitted to be designed using the safety factor, Ω , and resistance factor, ϕ , defined in Section C.1.2.2.

C.1.1.2 Elastic Buckling

Analysis shall be used for the determination of the elastic buckling loads and/or moments used in this Appendix. For columns, this includes the local, distortional, and overall buckling loads ($P_{cr\ell}$, P_{crd} , and P_{cre} of Section C.1.2.1). For beams, this includes the local, distortional, and overall buckling moments ($M_{cr\ell}$, M_{crd} , and M_{cre} of Section C.1.2.2). In some cases, for a given column or beam, all three modes do not exist. In such cases, the non-existent mode shall be ignored in the calculations of Sections C.1.2.1 and C.1.2.2. The commentary to this Appendix provides guidance on appropriate analysis procedures for elastic buckling determination.

C.1.1.3 Serviceability Determination

The bending deflection at any moment, M , due to nominal loads shall be permitted to be determined by reducing the gross moment of inertia, I_g , to an effective moment of inertia for deflection, as given in Eq. C.1-1:

$$I_{eff} = I_g(M_d/M) \leq I_g \quad (C.1-1)$$

where

M_d = Nominal flexural strength, M_n , defined in Section C.1.2.2, but with M_y replaced by M in all equations of Section C.1.2.2

M = Moment due to nominal loads on member to be considered ($M \leq M_y$)

C.1.2 Members

C.1.2.1 Column Design

The nominal axial strength [resistance], P_n , shall be the minimum of P_{ne} , $P_{nt\ell}$, and P_{nd} as given in Sections C.1.2.1.1 to C.1.2.1.3. For columns meeting the geometric and material criteria of Section C.1.1.1a, Ω_c and ϕ_c shall be as follows:

$$\Omega_c = 1.80(ASD) \quad \phi_c = 0.85(LRFD)$$

For all other columns, Ω and ϕ of the main Specification, Section 551.1.2(b), shall apply. The available strength shall be determined in accordance with applicable method in Section 551.4, or 551.5 of the main Specification.

C.1.2.1.1 Flexural, Torsional, or Flexural-Torsional Buckling

The nominal axial strength, P_{ne} , for flexural, torsional, or flexural-torsional buckling shall be calculated in accordance with the following:

- a. For $\lambda_c \leq 1.5$

$$P_{ne} = (0.658^{\lambda_c^2}) P_y \quad (C.1-2)$$

- b. For $\lambda_c \leq 1.5$

$$P_{ne} = \left(\frac{0.877}{\lambda_c^2} \right) P_y \quad (C.1-3)$$

where

$$\lambda_c = \sqrt{\frac{P_y}{P_{cre}}} \quad (C.1-4)$$

where

$$P_y = A_g F_y \quad (C.1-5)$$

P_{cre} = Minimum of the critical elastic column buckling load in flexural, torsional, or flexural-torsional buckling determined by analysis in accordance with Section C.1.1.2

C.1.1.1.2 Local Buckling

The nominal axial strength, $P_{n\ell}$, for local buckling shall be calculated in accordance with the following:

1. For $\lambda_l \leq 0.776$

$$P_{n\ell} = P_{ne} \quad (\text{C.1-6})$$

2. For $\lambda_l > 0.776$

$$P_{n\ell} = \left[1 - 0.15 \left(\frac{P_{crl}}{P_{ne}} \right)^{0.4} \right] \left(\frac{P_{crl}}{P_{ne}} \right)^{0.4} P_{ne} \quad (\text{C.1-7})$$

where

$$\lambda_l = \sqrt{\frac{P_{ne}}{P_{crl}}} \quad (\text{C.1-8})$$

P_{ne} = A value as defined in Section C.1.2.1.1

P_{crl} = Critical elastic local column buckling load determined by analysis in accordance with Section C.1.1.2

C.1.2.1c Distortional Buckling

The nominal axial strength, P_{nd} , for distortional buckling shall be calculated in accordance with the following:

1. For $\lambda_d \leq 0.561$

$$P_{nd} = P_y \quad (\text{C.1-9})$$

2. For $\lambda_d > 0.561$

$$P_{nd} = \left[1 - 0.25 \left(\frac{P_{crd}}{P_y} \right)^{0.6} \right] \left(\frac{P_{crd}}{P_y} \right)^{0.6} P_y \quad (\text{C.1-10})$$

where

$$\lambda_d = \sqrt{\frac{P_y}{P_{crd}}} \quad (\text{C.1-11})$$

where

P_y = A value as given in Eq. C.1-5

P_{crd} = Critical elastic distortional column buckling load determined by analysis in accordance with Section C.1.1.2

C.1.2.2 Beam Design

The nominal flexural strength, M_n , shall be the minimum of M_{ne} , $M_{n\ell}$, and M_{nd} as given in Sections C.1.2.2.1 to C.1.2.2.3. For beams meeting the geometric and material criteria of Section C.1.1.1.2, Ω_b and ϕ_b shall be as follows:

$$\Omega_b = 1.67(\text{ASD}) \quad \phi_b = 0.90(\text{LRFD})$$

For all other beams, Ω and ϕ of the main Specification, Section 551.1.2.(b), shall apply. The available strength [factored resistance] shall be determined in accordance with applicable method in Section 551.4, or 551.5 of the main Specification.

C.1.2.2.1 Lateral-Torsional Buckling

The nominal flexural strength, M_{ne} , for lateral-torsional buckling shall be calculated in accordance with the following:

1. For $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (\text{C.1-12})$$

2. For $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9} M_y \left(1 - \frac{10M_y}{36M_{cre}} \right) \quad (\text{C.1-13})$$

3. For $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (\text{C.1-14})$$

where

M_{cre} = Critical elastic lateral-torsional buckling moment determined by analysis in accordance with Section C.1.1.2

$$M_y = S_f F_y \quad (\text{C.1-15})$$

where

S_f = Gross section modulus referenced to the extreme fiber in first yield

C.1.2.2.2 Local Buckling

The nominal flexural strength, $M_{n\ell}$, for local buckling shall be calculated in accordance with the following:

1. For $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (\text{C.1-16})$$

2. For $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15 \left(\frac{M_{cr\ell}}{M_{ne}} \right)^{0.4} \right) \left(\frac{M_{cr\ell}}{M_{ne}} \right)^{0.4} M_{ne} \quad (\text{C.1-17})$$

where

$$\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}} \quad (\text{C.1-18})$$

M_{ne} = A value as defined in Section

C.1.2.2.1

Critical elastic local buckling moment

$M_{cr\ell}$ = determined by analysis in accordance with Section C.1.1.2

C.1.2.2.3 Distortional Buckling

The nominal flexural strength, M_{nd} , for distortional buckling shall be calculated in accordance with the following:

1. For $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (\text{C.1-19})$$

2. For $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22 \left(\frac{M_{crd}}{M_y} \right)^{0.5} \right) \left(\frac{M_{crd}}{M_y} \right)^{0.5} M_y \quad (\text{C.1-20})$$

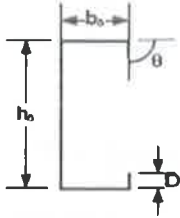
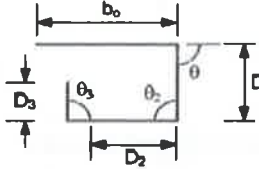
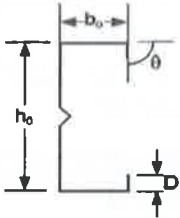
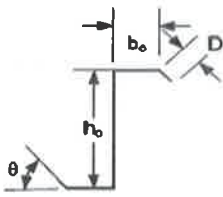
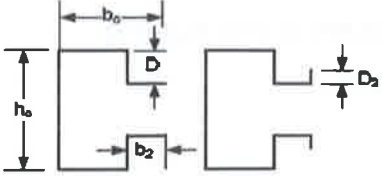
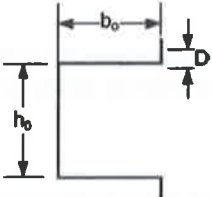
where

$$\lambda_d = \sqrt{M_y/M_{crd}} \quad (\text{C.1-21})$$

M_y = A value as given in Eq. C.1.2.2-4

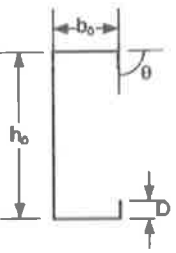
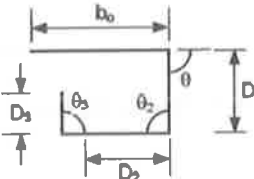
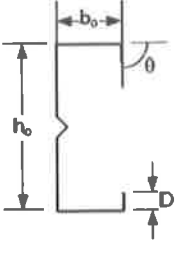
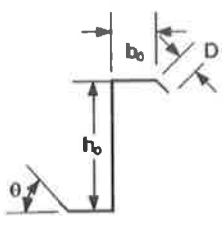
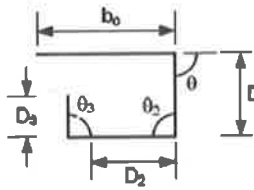
M_{crd} = Critical elastic distortional buckling moment determined by analysis in accordance with Section C.1.1.2

Table C-1
Limits for Pre-qualified Columns*

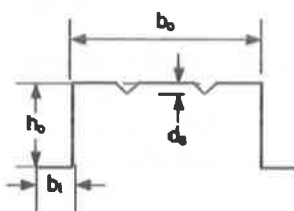
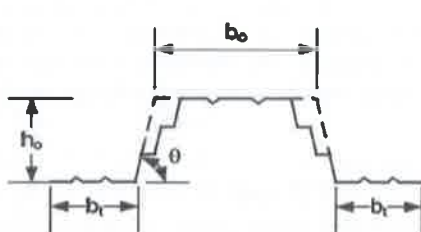
Lipped C-Sections Simple Lips:  Complex Lips: 	For all C-sections $h_o/t < 472$ $b_o/t < 159$ $4 < D/t < 33$ $0.7 < h_o/b_o < 5.0$ $0.05 < D/b_o < 0.41$ $0 - 90^\circ$ $E/F_y > 340 [F_y < 86 \text{ ksi (593 MPa or 6050 kg/cm}^2\text{)}]$ For C-sections with complete lips: $D_2/t < 34$ $D_2/D < 2$ $D_3/t < 34$ $D_3/D_2 < 1$ Note: a) θ_2 is permitted to vary (D_2 lip is permitted to angle inward, outward, etc.) b) θ_2 is permitted to vary (D_3 lip is permitted to angle up, down, etc.)
Lipped C-Sections with Web Stiffener (s) 	For one or two intermediate stiffeners: $h_o/t < 489$ $b_o/t < 160$ $6 < D/t < 33$ $1.3 < h_o/b_o < 2.7$ $0.05 < D/b_o < 0.41$ $E/F_y > 340 [F_y < 86 \text{ ksi (593 MPa or 6050 kg/cm}^2\text{)}]$
Z-Section 	$h_o/t < 137$ $b_o/t < 56$ $0 < D/t < 36$ $1.5 < h_o/b_o < 2.7$ $0.00 < D/b_o < 0.73$ $\theta - 50^\circ$ $E/F_y > 590 [F_y < 50 \text{ ksi (345 MPa or 3520 kg/cm}^2\text{)}]$
Rack Upright 	See C-Section with Complex Lips
Hat 	$h_o/t < 50$ $b_o/t < 43$ $4 < D/t < 6$ $1.0 < h_o/b_o < 1.2$ $D/b_o - 0.13$ $E/F_y > 428 [F_y < 69 \text{ ksi (476 MPa or 4850 kg/cm}^2\text{)}]$

Note: * $r/t < 10$, where r is the centerline bend radius
 b_o = overall width; D = overall lip depth; t = base metal thickness; h_o = overall depth

Table C-2
Limits for Pre-qualified Beams*

C-Sections Simple Lips:  Complex Lips: 	For all C-sections $h_o/t < 321$ $b_o/t < 75$ $0 < D/t < 34$ $1.5 < h_o/b_o < 17.0$ $0 < D/b_o < 0.70$ $44^\circ < \theta < 90^\circ$ $E/F_y > 421 [F_y < 70 \text{ ksi (483 MPa or 4920 kg/cm}^2\text{)}]$ For C-Sections with complex lips: $D_2/t < 34$ $D_2/D < 2$ $D_3/t < 34$ $D_3/D_2 < 1$ Note: θ_2 is permitted to vary (D_2 lip is permitted to angle inward or outward) θ_3 is permitted to vary (D_3 lip is permitted to angle up or down).
Lipped C-Sections with Web Stiffener 	$h_o/t < 358$ $b_o/t < 58$ $14 < D/t < 17$ $5.5 < h_o/b_o < 11.7$ $0.27 < D/b_o < 0.56$ $\theta < 90^\circ$ $E/F_y > 578 [F_y < 51 \text{ ksi (352 MPa or 3590 kg/cm}^2\text{)}]$
Z-Sections Simple Lips:  Complex Lips: 	For all Z-sections $h_o/t < 183$ $b_o/t < 71$ $10 < D/t < 16$ $2.5 < h_o/b_o < 4.1$ $0.15 < D/b_o < 0.34$ $36^\circ < \theta < 90^\circ$ $E/F_y > 440 [F_y < 67 \text{ ksi (462 MPa or 4710 kg/cm}^2\text{)}]$ For Z-Sections with complex lips: $D_2/t < 34$ $D_2/D < 2$ $D_3/t < 34$ $D_3/D_2 < 1$ Note: θ_2 is permitted to vary (D_2 lip is permitted to angle inward or outward) θ_3 is permitted to vary (D_3 lip is permitted to angle up, down, etc.)

Limitations for Pre-qualified Beams (Continued)

Hat (Decks) with Stiffened Flange in Compression 	$h_o/t < 97$ $b_o/t < 467$ $0 < d_s/t < 26 \quad (d_s = \text{Depth of stiffener})$ $0.14 < h_o/b_o < 0.87$ $0.88 < b_o/b_t < 5.4$ $0 < n \leq 4 \quad (n = \text{Number of compression flange stiffeners})$ $E/F_y > 440 [F_y < 67 \text{ ksi (462 MPa or 4710 kg/cm}^2\text{)}]$
Trapezoids (Decks) with Stiffened Flange in Compression 	$h_o/t < 203$ $b_o/t < 231$ $0.42 < h_o/\sin \theta / b_o < 1.91$ $1.10 < b_o/b_t < 3.38$ $0 < n_c \leq 2 \quad (n_c = \text{Number of compression flange stiffeners})$ $0 < n_w \leq 2 \quad (n_w = \text{Number of web stiffeners and /or folds})$ $0 < n_t \leq 2 \quad (n_t = \text{Number of tension flange stiffeners})$ $< n_t \leq 2 \quad (n_t = \text{Number of tension flange stiffeners})$ $52^\circ < \theta < 84^\circ \quad (\theta = \text{Angle between web and horizontal plane})$ $E/F_y > 310 [F_y < 95 \text{ ksi (655 MPa or 6680 kg/cm}^2\text{)}]$

Note: $r/t < 10$, where r is the centerline bend radius.

See Section C.1.1.1 for definitions of other variables given in Table C-1 and Table C-2.

SECTION C-2

SECOND-ORDER ANALYSIS

This Section C.2 addresses second-order analysis for structural systems comprised of moment frames, braced frames, shear walls, or combinations thereof.

C.2.1 General Requirements

Members shall satisfy the provisions of Section 553.5 with the nominal column strengths [nominal axial resistance], P_n , determined using K_x and $K_y = 1.0$, as well as $\alpha x = 1.0$, $\alpha_y = 1.0$, $C_{mx} = 1.0$, and $C_{my} = 1.0$. The required strengths [factored forces and moments] for members, connections, and other structural elements shall be determined using a second-order analysis as specified in this Section. All component and connection deformations that contribute to the lateral displacement of the structure shall be considered in the analysis.

C.2.2 Design and Analysis Constraints

C.2.2.1 General

The second-order analysis shall consider both the effect of loads acting on the deflected shape of a member between joints or nodes ($P - \delta$ effects) and the effect of loads acting on the displaced location of joints or nodes in a structure ($P - \Delta$ effects). It shall be permitted to perform the analysis using any general second-order analysis method. Analyses shall be conducted according to the design and loading requirements specified in Section 551. For the ASD, the second-order analysis shall be carried out under 1.6 times the ASD load combinations and the results shall be divided by 1.6 to obtain the required strengths at allowable load levels.

C.2.2.2 Types of Analysis

It shall be permissible to carry out the second-order analysis either on the out-of-plumb geometry without notional loads or on the plumb geometry by applying notional loads or minimum lateral loads as defined in Section C.2.2.4.

For second-order elastic analysis, axial and flexural stiffness shall be reduced as specified in Section C.2.2.3.

C.2.2.3 Reduced Axial and Flexural Stiffnesses

Flexural and axial stiffness shall be reduced by using E^* in place of E as follows for all members whose flexural and axial stiffnesses are considered to contribute to the lateral stability of the structure:

$$E^* = 0.8\tau_b E \quad (C.2-1)$$

where

$$\begin{aligned} \tau_b &= 1.0 \text{ for } \alpha P_r / P_y \leq 0.5 \\ &= 4[\alpha P_r / P_y (1 - \alpha P_r / P_y)] \text{ for } \\ &\quad \alpha P_r / P_y > 0.5 \end{aligned}$$

P_r = Required axial compressive strength [factored axial compressive force], (N)

P_y = Member yield strength ($= A F_y$, where A is the full unreduced cross-sectional area), (N)

$$\begin{aligned} \alpha &= 1.6 \text{ (ASD)} \\ &= 1.0 \text{ (LRFD)} \end{aligned}$$

In cases where flexibility of other structural components such as connections, flexible column base details, or horizontal trusses acting as diaphragms is modeled explicitly in the analysis, the stiffnesses of the other structural components shall be reduced by a factor of 0.8.

If notional loads are used, in lieu of using $\tau_b < 1.0$ where $\alpha P_r / P_y > 0.5$, $\tau_b = 1.0$ shall be permitted to be used for all members, provided that an additional notional load of $0.001Y_i$ is added to the notional load required in Section C.2.2.4.

C.2.2.4 Notional loads

Notional loads shall be applied to the lateral framing system to account for the effects of geometric imperfections. Notional loads are lateral loads that are applied at each framing level and specified in terms of the gravity loads applied at that level. The gravity load used to determine the notional load shall be equal to or greater than the gravity load associated with the load combination being evaluated. Notional loads shall be applied in the direction that adds to the destabilizing effects under the specified load combination.

A notional load, $N_i = (1/240) Y_i$, shall be applied independently in two orthogonal directions as a lateral load in all load combinations. This load shall be in addition to other lateral loads, if any.

N_i = Notional lateral load applied at level i , kips (N)

Y_i = Gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i , N

The notional load coefficient of 1/240 is based on an assumed initial story out-of-plumbness ratio of 1/240. Where a different assumed out-of-plumbness is justified, the notional load coefficient shall be permitted to be adjusted proportionally to a value not less than 1/500.

SECTION C3

ADDITIONAL PROVISIONS

This Section provides design provisions or supplements to Section 551 through 557.

C.3.1 Scope

Designs shall be made in accordance with the provisions for Load and Resistance Factor Design, or with the provisions for Allowable Strength Design.

C.3.2 Other Steels

The listing in Section C.3.1 shall not exclude the use of steel up to and including 25 mm in thickness, ordered or produced to other than the listed specifications, provided the following requirements are met:

1. The steel shall conform to the chemical and mechanical requirements of one of the listed specifications or other published specification.
2. The chemical and mechanical properties shall be determined by the producer, the supplier, or the purchaser, in accordance with the following specifications. For coated sheets, ASTM A924/ A924M; for hot-rolled or cold-rolled sheet and strip, ASTM A568/ A568M; for plate and bar, ASTM A6/ A6M; for hollow structural sections, such tests shall be made in accordance with the requirements of A500 (for carbon steel) or A847 (for HSLA steel).
3. The coating properties of coated sheet shall be determined by the producer, the supplier, or the purchaser, in accordance with ASTM A924/ A924M.
4. The steel shall meet the requirements of Section C.3.3.
5. If the steel is to be welded, its suitability for the intended welding process shall be established by the producer, the supplier, or the purchaser in accordance with AWS D1.1 or D1.3 as applicable.

If the identification and documentation of the production of the steel have not been established, then in addition to requirements (1) through (5), the manufacturer of the cold-formed steel product shall establish that the yield stress and tensile strength of the master coil are at least 10 percent greater than specified in the referenced published specification.

C.3.2.1 Ductility

In seismic design category D, E or F (as defined by ASCE/SEI 7), when material ductility is determined on the basis of the local and uniform elongation criteria of Section C.3.3.1, curtain wall studs shall be limited to the dead load of the curtain wall assembly divided by its surface area, but no greater than 0.75 kN/m^2 .

C.3.3 Loads

C.3.3.1 Nominal Loads

The nominal loads shall be as stipulated by the applicable building code under which the structure is designed or as dictated by the conditions involved. In the absence of a building code, the nominal loads shall be those stipulated in the ASCE/SEI 7.

C.3.3.1.1 Load Combinations for ASD

The structure and its components shall be designed so that the allowable strengths equal or exceed the effects of the nominal loads and load combinations as stipulated by the applicable building code under which the structure is designed or, in the absence of an applicable building code, as stipulated in the ASCE/SEI 7.

C.3.3.1.2 Load Factors and Load Combinations for LRFD

The structure and its components shall be designed so that design strengths equal or exceed the effects of the factored loads and load combinations stipulated by the applicable building code under which the structure is designed or, in the absence of an applicable building code, as stipulated in the ASCE/SEI 7.

C.3.4 Referenced Documents

The following documents are referenced in Section C-3:

1. American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 700, Chicago, Illinois 60601-1802: ANSI/ AISC 360-05, Specification for Structural Steel Buildings
2. American Iron and Steel Institute (AISI), 1140 Connecticut Avenue, NW, Washington, DC 20036:

AISI S213-07, North American Standard for Cold-Formed Steel Framing – Lateral Design
AISI S908-04, Base Test Method for Purlins Supporting a standing Seam Roof System

3. American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston VA, 20191: ASCE/SEI 7-05, Minimum Design Loads in Buildings and Other Structures
4. American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135: AWS D1.3-98, Structural Welding Code-Sheet Steel AWS C1.1/C1.1M-2000, Recommended Practices for Resistance Welding

C.3.5 Tension Members

For axially loaded tension members, the nominal tensile strength, T_n , shall be the smallest value obtained in accordance with the limit states of (a), (b) and (c). Unless otherwise specified, the corresponding safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section 551.4 or 551.5.

1. For yielding in gross section

$$T_n = A_g F_y \quad (C.3-1)$$

$$\Omega_t = 1.67(ASD) \quad \phi = 0.90(LRFD)$$

where

T_n = Nominal strength of member when loaded in tension

A_g = Gross area of cross section

F_y = Design yield stress as determined in accordance with Section 551.7.1

2. For rupture in net section away from connection

$$T_n = A_n F_u \quad (C.3-2)$$

$$\Omega_t = 2.00(ASD) \quad \phi_t = 0.75(LRFD)$$

where

A_n = Net area of cross section

F_u = Tensile strength as specified in either Section 551.2.1 or 551.2.3.2

3. For rupture in net section at connection

The available tensile strength shall also be limited by Sections 555.2.7, 555.3, and 555.5 for tension members using welded connections, bolted connections, and screw connections.

C.3.6 Light-Frame Steel Construction

In addition to the cold-formed steel framing standards listed in Section 554.4, the following standard shall be followed, as applicable:

1. Light-framed shear walls, diagonal strap bracing (that is part of a structural wall) and diaphragms to resist wind, seismic and other in-plane lateral loads shall be designed in accordance with AISI S213.

C.3.6.1 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

The available flexural strength of a C- or Z-section, loaded in a plane parallel to the web with the top flange supporting a standing seam roof system shall be determined using discrete point bracing and the provisions of Section 553.3.1.2.1, or shall be calculated in accordance with this section. The safety factor and the resistance factor provided in this section shall be applied to the nominal strength, M_n , calculated by Eq. 554.6.1.2-1 to determine the available strengths in accordance with the applicable method in Section 551.4 or 551.5.

$$M_n = R S_e F_y \quad (C.3-3)$$

$$\Omega_b = 1.67(ASD) \quad \phi_b = 0.90(LRFD)$$

where

R = Reduction factor determined in accordance with AISI S908

See Section 553.3.1.1 for definitions of S_e and F_y .

C.3.6.2 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

These provisions shall apply to Z-sections concentrically loaded along their longitudinal axis, with only one flange attached to standing seam roof panels. Alternatively, design values for a particular system shall be permitted to be based on discrete point bracing locations, or on tests in accordance with Section 556.

The nominal axial strength of simple span or continuous Z-sections shall be calculated in accordance with (a) and (b). Unless otherwise specified, the safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section 551.4 or 551.5.

1. For weak axis available strength

$$P_n = k_{af} R F_y A \quad (\text{C.3-4})$$

$$\Omega = 1.80 \text{ (ASD)} \quad \phi = 0.85 \text{ (LRFD)}$$

where

- a. For $d/t \leq 90$

$$k_{af} = 0.36$$

- b. For $90 < d/t \leq 130$

$$k_{af} = 0.72 - \frac{d}{250t} \quad (\text{C.3-5})$$

- c. For $d/t \leq 130$

$$k_{af} = 0.20$$

R = Reduction factor determined from uplift tests performed using AISI S908

A = Full unreduced cross-sectional area of Z-section

d = Z-section depth

t = Z-section thickness

See Section 553.3.1.1 for definition of F_y .

Eq. 554.6.1-4-1 shall be limited to roof systems meeting the following conditions:

- Purlin thickness, $1.37 \text{ mm} \leq t \leq 3.22 \text{ mm}$
 - $150 \text{ mm} \leq d \leq 300 \text{ mm}$
 - Flanges are edge stiffened compression elements
 - $70 \leq d/t \leq 170$
 - $2.8 \leq d/b < 5$, where b = Z section flange width.
 - $16 \leq \frac{\text{flange flat width}}{t} < 50$
 - Both flanges are prevented from moving laterally at the supports
 - Yield stress, $F_y \leq 483 \text{ MPa}$
2. The available strength about the strong axis shall be determined in accordance with Section 553.4.1 and 553.4.1.1.

C.3.6.3 Strength of Standing Seam Roof Panel Systems

In addition to the provisions provided in Section 554.6.2.1, for load combinations that include wind uplift, the nominal wind load shall be permitted to be multiplied by 0.67 provided the tested system and wind load evaluation satisfies the following conditions:

- The roof system is tested in accordance with AISI S906.
- The wind load is calculated using ASCE/SEI 7 for components and cladding, Method 1 (Simplified Procedure) or Method 2 (Analytical Procedure).
- The area of the roof being evaluated is in Zone 2 (edge zone) or Zone 3 (corner zone), as defined in ASCE/SEI 7, i.e. the 0.67 factor does not apply to the field of the roof (Zone 1).
- The base metal thickness of the standing seam roof panel is greater than or equal to 0.60 mm and less than or equal to 0.80 mm.
- For trapezoidal profile standing seam roof panels, the distance between sidelaps is no greater than 600 mm.
- For vertical rib profile standing seam roof panels, the distance between sidelaps is no greater than 450 mm.
- The observed failure mode of the tested system is one of the following:
 - The standing seam roof clip mechanically fails by separating from the panel sidelap
 - The standing seam roof clip mechanically fails by the sliding-tab separating from the stationary base.

C.3.7 Welded Connections

Welded connections in which the thickness of the thinnest connected part is greater than 5 mm shall be in accordance with ANSI/AISC-360.

Except as modified herein, arc welds on steel where at least one of the connected parts is 5 mm or less in thickness shall be made in accordance with AWS D1.3. Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions are intended to cover the welding positions as listed in Table C.3.1.

Resistance welds shall be made in conformance with the procedures given in AWS C1.1 or AWS C1.3.

Table C.3-1 Welding Position Covered

Connection	Welding Position					
	Square Groove Butt Weld	Arc Spot Weld	Arc Seam Weld	Fillet Weld, Lap or T	Flare Bevel Groove	Flare V Groove Weld
Sheet to sheet	F	-	F	F	F	F
	H	-	H	H	H	H
	V	-	-	V	V	V
Sheet to Supporting Member	OH	-	-	OH	OH	OH
	-	F	F	F	F	-
	-	-	-	H	H	-
	-	-	-	V	V	-
	-	-	-	OH	OH	-

(F = flat, H = horizontal, V = vertical, OH = over head)

C.3.8 Bolted Connections

In addition to the design criteria given in Section C3.8 of this Specification, the following design requirements shall also be followed for bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than 4.76 mm. Bolted connections in which the thickness of the thinnest connected part is equal to or greater than 4.76 mm shall be in accordance with ANSI/AISC-360.

The holes for bolts shall not exceed the sizes specified in Table C.3-2, except that larger holes are permitted to be used in column base details or structural systems connected to concrete walls.

Standard holes shall be used in bolted connections, except that oversized and slotted holes shall be permitted to be used as approved by the designer. The length of slotted holes shall be normal to the direction of the shear load. Washers or backup plates shall be installed over oversized or slotted holes in an outer ply unless suitable performance is demonstrated by tests in accordance with Section 556. In the situation where the holes occurs within the lap of lapped and nested zee members, the above requirements regarding the direction of the slot and the use of washers shall be permitted not to apply, subject to the following limits:

1. 12.7 mm diameter bolts only,
2. Maximum slot size is 14.3 mm x 22.2 mm slotted vertically,
3. Maximum oversize hole is 16 mm diameter,
4. Minimum member thickness is 1.52 mm nominal,
5. Maximum member yield stress 410 MPa,
6. Minimum lap length measured from center of frame to end of lap is 1.5 times the member depth.

Table C.3-2 Maximize Size of Bolt Holes, millimeters

Nominal Bolt Diameter, d mm	Standard Hole Diameter dh mm	Oversized Hole Diameter, dh mm	Short Slotted Hole Dimensions mm	Long Slotted Hole Dimensions mm
< 12.7	$d + 0.8$	$d + 1.6$	$(d + 0.8)$ by $(d + 6.4)$	$(d + 0.8)$ by $(21/2 d)$
≥ 12.7	$d + 1.6$	$d + 3.2$	$(d + 1.6)$ by $(d + 6.4)$	$(d + 1.6)$ by $(21/2 d)$

C.3.8.1 Shear, Spacing and Edge Distance

The nominal shear strength, P_n , of the connected part as affected by spacing and edge distance in the direction of applied force shall be calculated in accordance with Eq.C3-6. The corresponding safety factor and the resistance factor provided in this section shall be used to determine the available strength in accordance with the applicable method in Section 551.4 or 551.5.

$$P_n = t_e F_u \quad (C3-6)$$

- a. When $F_u/F_{sy} \geq 1.08$

$$\Omega = 2.00 \text{ (ASD)} \quad \phi = 0.70 \text{ (LRFD)}$$

- b. When $F_u/F_{sy} < 1.08$

$$\Omega = 2.22 \text{ (ASD)} \quad \phi = 0.60 \text{ (LRFD)}$$

where

P_n = Nominal strength per bolt

e = Distance measured in line of force from center of a standard hole to nearest edge of a adjacent hole or to end of connected part

t = Thickness of thinnest connected part

F_u = Tensile strength of connected part as specified in Section 551.2.1, 551.2.2 or 551.2.3

F_{sy} = Yield stress of connected part as specified in Section 551.2.1, 551.2.2 or 551.2.3

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench but shall not be less than 3 times the nominal bolt diameter, d . also, the distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than $1\frac{1}{2} d$.

For oversized and slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of $e - (dh/2)$, in which e is the required distance used in Eq. C.3-6, and dh is the diameter of a standard hole defined in Table C.3-2. In no case shall the clear distance between edges of two adjacent holes be less than $2d$ and the distance between the edge of the hole and the end of the member be less than d .

C.3.8.2 Rupture in Net Section (Shear Lag)

The nominal tensile strength of a bolted member shall be determined in accordance with Section 553. For rupture in the effective net section of the connected part, the nominal tensile strength, P_n shall be determined in accordance with this section. Unless otherwise specified, the corresponding safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section 551.4 or 551.5.

- a. For flat sheet connections not having staggered hole patterns

$$P_n = A_n F_t \quad (C.3-7)$$

1. When washers are provided under both the bolt head and the nut

For single bolt, or a single row of bolts perpendicular to the force

$$F_t = (0.1 + 3d/s)F_u \leq F_u \quad (C.3-8)$$

For multiple bolts in the line parallel to the force

$$F_t = F_u \quad (C.3-9)$$

For double shear:

$$\Omega = 2.00 \text{ (ASD)} \quad \phi = 0.65 \text{ (LRFD)}$$

For single shear:

$$\Omega = 2.22 \text{ (ASD)} \quad \phi = 0.55 \text{ (LRFD)}$$

2. When either washers are not provided under the bolt head and the nut, or only one washer is provided under either the bolt head or the nut

For single bolt, or a single row of bolts perpendicular to the force

$$F_t = (2.5 + d/s)F_u \leq F_u \quad (C.3-10)$$

For multiple bolts in the line parallel to the force

$$F_t = F_u \quad (C.3-11)$$

$$\Omega = 2.22 \text{ (ASD)} \quad \phi = 0.65 \text{ (LRFD)}$$

where

- A_n = Net Area of connected part
 F_t = Nominal Tensile stress in flat sheet
 d = Nominal bolt diameter
 s = Sheet width divided by number of bolt holes in cross section being analyzed (when evaluating F_t)
 F_u = Tensile strength of connected part as specified in Section 551.2.1, 551.2.2 or 551.2.3

- b. For flat sheet connections having staggered hole patterns

$$P_n = A_n F_t \quad (C.3-12)$$

$$\Omega = 2.22 \text{ (ASD)} \quad \phi = 0.65 \text{ (LRFD)}$$

where

- F_t = Determined in accordance with Eqs. E3.2-2 to E3.2-5.
 $A_n = 0.90[A_g - n_b d_h t + (\sum s'^2/4g)t]$ (C.3-13)

- A_g = Gross area of member
 s' = Longitudinal center-to-center spacing of any two consecutive holes
 g = Transverse center-to-center spacing between fastener gauge lines
 n_b = Number of bolt holes in the cross section being analyzed
 d_h = Diameter of a standard hole

See Section C.3.8.1 for the definition of t .

- (c) For other than flat sheet

$$P_n = A_e F_u \quad (C.3-14)$$

$$\Omega = 2.22 \text{ (ASD)} \quad \phi = 0.65 \text{ (LRFD)}$$

where

- $A_e = A_n U$, effective net area with U defined as follows:
 $U = 1.0$ for members when the load is transmitted directly to all of the cross-sectional elements.
 Otherwise, the reduction coefficient U is

determined as follows:

1. For Angle members having two or more bolts in the line of force

$$U = 1.0 - 1.20 x/L < 0.9 \quad (\text{C.3-15})$$

$$\text{But } U \geq 0.4$$

2. For channel members having two or more bolts in the line of force

$$U = 1.0 - 0.36 x/L < 0.9 \quad (\text{C.3-16})$$

$$\text{but } U \geq 0.5$$

where

x = Distance from shear plane to centroid of the cross

L = Length of connection

C.3.8.3 Shear and Tension in Bolts

The nominal bolt strength, P_n , resulting from shear, tension or of combination of shear and tension shall be calculated in accordance with this section. The corresponding safety factor and the resistance factor provided in Table C.3-3 shall be used to determine the available strengths in accordance with the applicable method in Section 551.4 or 551.5.

$$P_n = A_b F_n \quad (\text{C.3-17})$$

where

A_b = Gross cross-sectional area of bolt

F_n = Nominal strength ksi (MPa) is determined in accordance with (a) or (b) as follows:

- (a) When bolts are subjected to shear only or tension only F_n shall be given by F_{nv} or F_{nt} in Table C.3-3.

Corresponding safety and resistance factor, Ω and ϕ , shall be accordance with Table C.3-3.

The pullover strength of the connected sheet at the bolt head, nut or washer shall be considered where bolt tension is involved. See Section 555.6.

- (b) When bolts are subjected to a combination of shear and tension, F_n , is given by F'_{nt} in Eq.C.3-18 or C.3-19 as follows

For ASD

$$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \leq F_{nt} \quad (\text{C.3-18})$$

For LRFD

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt} \quad (\text{C.3-19})$$

where

F'_{nt} = Nominal tensile stress modified to include the effects of required shear stress, MPa

F_{nt} = Nominal tensile stress from Table C.3-3

F_{nv} = Required shear stress, MPa

Ω = Safety factor for shear from Table C.3-3

ϕ = Resistance factor for shear from Table C.3-3

In addition, the required shear stress, f_v , shall not exceed the allowable shear stress, $F_{nv}/\Omega(\text{ASD})$ or the design shear stress, $\phi F_{nv}(\text{LRFD})$, of the fastener.

In Table C.3-3, the shear strength shall apply to bolts in holes as limited by Table C.3-2. Washers or back-up plates shall be installed over long-slotted holes and the capacity of connections using long-slotted holes shall be determined by load tests in accordance with Section 556.

C.3.8.3.1a Connection Shear Limited by End Distance

The nominal shear strength per screw, P_{ns} shall not exceed that calculated in accordance with Eq. C.3-20 where the distance to an end of the connected part is parallel to the line of the applied force. The safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section 551.4 or 551.5.

$$P_{ns} = teF_u \quad (\text{C.3-20})$$

$$\Omega = 3.00(\text{ASD}) \quad \phi = 0.50(\text{LRFD})$$

where

t = Thickness of part in which end distance is measured

e = Distance measured in line of force from center of a standard hole to nearest end of connected part

F_u = Tensile strength of part in which end distance is measured

Table C.3-3 Nominal Tensile and Shear Strengths for Bolts

Bolts	Tensile Strength			Shear Strength		
	Safety Factor Ω (ASD)	Resistance Factor ϕ (LRFD)	Nominal Stress F_{nt} Mpa	Safety Factor Ω (ASD)	Resistance Factor ϕ (LRFD)	Nominal Stress F_{nv} Mpa
A307 Bolts Grade A 6.4 mm ≤ d < 12.7 mm	2.25	0.75	279	2.4	0.65	165
A307 Bolts Grade A d ≥ 12.7 mm	2.25		310			186
A325 Bolts, when threads are not excluded from shear planes	2.00		621			372
A325 Bolts, when threads are excluded from shear planes			621			496
A354 Grade BD Bolts 6.4 mm ≤ d < 12.7 mm, when threads are not excluded from shear planes			696			407
A354 Grade BD Bolts 6.4 mm ≤ d < 12.7 mm, when threads are excluded from shear planes			696			621
A449 Bolts 6.4 mm ≤ d < 12.7 mm, when threads are not excluded from shear planes			558			324
A449 Bolts 6.4 mm ≤ d < 12.7 mm; when threads are excluded from shear planes			558			496
A490 Bolts when threads are not excluded from shear planes			776			465
A490 Bolts when threads are not excluded from shear planes			776			621

C.3.9 Rupture**C.3.9.1 Shear Rupture**

At beam-end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the nominal shear strength, V_n , shall be calculated in accordance with Eq. C.3-21. The safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section 551.4 or 551.5.

$$V_n = 0.6F_uA_{wn} \quad (C.3-21)$$

$$\Omega = 2.00(ASD) \quad \phi = 0.75(LRFD)$$

where

- A_{wn} = $(h_{wc} - nd_h)t$
- h_{wc} = Coped flat web depth
- n = Number of holes in critical plane
- d_h = Hole diameter
- F_u = Tensile strength of connected part as specified in Section 551.2.1 or 551.2.2
- t = Thickness of coped web

C.3.9.2 Tension Rupture

The available tensile strength along a path in the affected elements of connected members shall be determined by Section 555.2.7 or 555.3.2 for welded or bolted connections, respectively.

C.3.9.3 Block Shear Rupture

When the thickness of the thinnest connected part is less than 4.76 mm, the block shear rupture nominal strength, R_n , shall be determined in accordance with this section. Connections in which the thickness of the thinnest connected part is equal to or greater than 4.76 mm shall be in accordance with ANSI/ AISC-360.

The nominal block shear rupture strength, R_n , shall be determined as the lesser of Eqs. C.3-22 and C.3-23. The corresponding safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section 551.4 or 551.5.

$$R_n = 0.6F_yA_{gv} + F_uA_{nt} \quad (C.3-22)$$

$$R_n = 0.6F_yA_{nv} + F_uA_{nt} \quad (C.3-23)$$

For bolted connections

$$\Omega = 2.22(ASD) \quad \phi = 0.65(LRFD)$$

For welded connections

$$\Omega = 2.50(ASD) \quad \phi = 0.60(LRFD)$$

where

- A_{gv} = Gross area subject to shear
- A_{nv} = Net area subject to shear
- A_{nt} = Net area subject to tension

Chapter 6

WOOD

NATIONAL STRUCTURAL CODE OF THE PHILIPPINES VOLUME I BUILDINGS, TOWERS AND OTHER VERTICAL STRUCTURES

SEVENTH EDITION, 2015

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Table of Contents

SECTION 601 - GENERAL REQUIREMENTS.....	4
601.1 Scope.....	4
601.2 Design Method.....	4
SECTION 602 - NOTATIONS AND DEFINITIONS	4
602.1 Notations	4
602.2 Definitions.....	7
SECTION 603 - MINIMUM QUALITY	8
603.1 Quality and Identification	8
603.2 Minimum Capacity or Grade.....	8
603.3 Timber Connectors and Fasteners.....	8
603.4 Fabrication, Installation and Manufacture.....	8
SECTION 604 - DESIGN AND CONSTRUCTION REQUIREMENTS	9
604.1 General.....	9
PART I - REQUIREMENTS APPLICABLE TO ALL DESIGN PROCEDURES.....	10
SECTION 605 - DECAY AND TERMITE PROTECTION.....	10
605.1 Preparation of Building Site.....	10
605.2 Wood Support Embedded in Ground	10
605.3 Under-Floor Clearance.....	10
605.4 Plates, Sills and Sleepers.....	10
605.5 Columns and Posts	10
605.6 Girders Entering Masonry or Concrete Walls	10
605.7 Under-Floor Ventilation.....	10
605.8 Wood and Earth Separation.....	11
605.9 Wood Supporting Roofs and Floors.....	11
605.10 Moisture Content of Treated Wood	11
605.11 Retaining Walls.....	11
605.12 Weather Exposure	11
605.13 Water Splash	11
SECTION 606 - WOOD SUPPORTING MASONRY OR CONCRETE	12
606.1 Dead Load.....	12
606.2 Horizontal Force	12
SECTION 607 - WALL FRAMING	12
SECTION 608 - FLOOR FRAMING	13
SECTION 609 - EXTERIOR WALL COVERINGS.....	14
609.1 General.....	14
609.2 Siding	14
609.3 Plywood	15
609.4 Shingles or Shakes	15
609.5 Particleboard	15
609.6 Hardboard.....	15
609.7 Nailing.....	15

SECTION 610 - INTERIOR PANELING	18
SECTION 611 - SHEATHING	18
611.1 Structural Floor Sheathing.....	18
611.2 Structural Roof Sheathing	18
SECTION 612 - MECHANICALLY-LAMINATED FLOORS AND DECKS	22
SECTION 613 - POST-BEAM CONNECTIONS	22
PART II - REQUIREMENTS APPLICABLE TO DESIGN OF WIND AND EARTHQUAKE LOAD-RESISTING SYSTEMS.....	23
SECTION 614 - WOOD SHEAR WALLS AND DIAPHRAGMS.....	23
614.1 General	23
614.2 Wood Members Resisting Horizontal Forces Contributed by Masonry and Concrete.....	24
614.3 Wood Diaphragms.....	24
614.4 Particleboard Diaphragms	25
614.5 Wood Shear Walls and Diaphragms in Seismic Zone 4.....	25
614.6 Fiberboard Sheathing Diaphragms	26
SECTION 615 - DESIGN VALUES FOR STRUCTURAL MEMBERS.....	27
615.1 General	27
615.2 Reference Design Values	27
615.3 Adjustment of Reference Design Values.....	27
SECTION 616 - DESIGN PROVISIONS AND EQUATIONS.....	40
616.1 General	40
616.2 Bending Members - General	41
616.3 Bending Members - Flexure	41
616.4 Bending Members - Shear	43
616.5 Bending Members - Deflection	45
616.6 Compression Members - General	45
616.7 Solid Columns	46
616.8 Tension Members	47
616.9 Combined Bending and Axial Loading	47
616.10 Design for Bearing	48
SECTION 617 - SAWN LUMBER.....	50
617.1 General	50
617.2 Reference Design Values	51
617.3 Adjustment of Reference Design Values.....	52
617.4 Special Design Considerations.....	54
SECTION 618 - STRUCTURAL GLUED LAMINATED TIMBER.....	56
618.1 General	56
618.2 Reference Design Values	56
618.3 Adjustment of Reference Design Values.....	58
618.4 Special Design Considerations	60
SECTION 619 - TIMBER CONNECTORS AND FASTENERS	63
619.1 General	63
619.2 Bolts	63
619.3 Nails and Spikes	68
619.4 Joist Hangers and Framing Anchors.....	71
619.5 Miscellaneous Fasteners.....	71

SECTION 620 - CONVENTIONAL LIGHT-FRAME CONSTRUCTION DESIGN PROVISIONS.....	73
620.1 General.....	73
620.2 Design of Portions.....	73
620.3 Additional Requirements for Conventional Construction in High-wind Areas.....	73
620.4 Additional Requirements for Conventional Construction in Seismic Zone 2.....	73
620.5 Girders.....	74
620.6 Floor Joists.....	74
620.7 Subflooring.....	75
620.8 Particleboard Underlayment.....	75
620.9 Wall Framing.....	75
SECTION 621 - METAL PLATE-CONNECTED WOOD TRUSS DESIGN.....	83
621.1 Design and Fabrication.....	83
621.2 Performance.....	83
621.3 In-Plant Inspection.....	83
621.4 Marking.....	83
SECTION 622 - USE OF MACHINE-GRADED LUMBER (MGL).....	83
622.1 General.....	83
622.2 Design Properties for Machine-Graded Lumber.....	83
622.3 Design Using Machine-Graded Lumber.....	83
622.4 Preservative Treatment.....	83
622.5 Moisture Content.....	83
622.6 Markings.....	83

SECTION 601 GENERAL REQUIREMENTS

601.1 Scope

The provisions of this chapter shall govern the materials, design, construction, and quality of wood members and their fasteners.

601.2 Design Method

Design shall be based on one of the following methods:

601.2.1 Working Stress Design (WSD)

Design using working stress design methods shall resist the different load combinations in accordance with the applicable requirements of Section 604.

601.2.2 Conventional Light-Frame Construction

The design and construction of conventional light-frame wood structures shall be in accordance with the applicable requirements of Section 604 and the NSCP Volume III - Housing.

SECTION 602 NOTATIONS AND DEFINITIONS

602.1 Notations

Except where otherwise noted, the symbols used in this Chapter have the following meanings:

A	=	area of cross section, mm^2
A_n	=	cross-sectional area of notched member, mm^2
C_D	=	load duration factor
C_F	=	size factor for sawn lumber
C_I	=	stress interaction factor for tapered glued laminated timbers
C_L	=	beam stability factor
C_M	=	wet service factor
C_P	=	column stability factor
C_T	=	buckling stiffness factor for dimension lumber
C_V	=	volume factor for structural glued laminated timber
C_b	=	bearing area factor
C_c	=	curvature factor for structural glued laminated timber
C_{dt}	=	empirical constant derived from relationship of equations for deflection of tapered straight beams and prismatic beams
C_{fu}	=	flat use factor
C_i	=	incising factor for dimension lumber
C_r	=	repetitive member factor for dimension lumber
C_{rs}	=	empirical load-shape radial stress reduction factor for double-tapered curved structural glued laminated timber bending members
C_t	=	temperature factor
C_{vr}	=	shear reduction factor for structural glued laminated timber
C_y	=	tapered structural glued laminated timber beam deflection factor
COV_E	=	coefficient of variation for modulus of elasticity

E, E'	= reference and adjusted modulus of elasticity, MPa	F_c, F_c'	= reference and adjusted compression design value parallel to grain, MPa
E_{axial}	= modulus of elasticity of structural glued laminated timber for extensional deformations, MPa	F_c^*	= reference compression design value parallel to grain multiplied by all applicable adjustment factors except C_P , MPa
E_{min}, E_{min}'	= reference and adjusted modulus of elasticity for beam stability and column stability calculations, MPa	F_{cE}	= critical buckling design value for compression members, MPa
$(EI)_{min}, (EI)_{min}'$	= reference and adjusted EI for beam stability and column stability calculations, MPa	F_{cE1}, F_{cE2}	= critical buckling design value for compression member in planes of lateral support, MPa
E_x	= modulus of elasticity of structural glued laminated timber for deflections due to bending about the x-x axis, MPa	$F_{c\perp}, F_{c\perp}'$	= reference and adjusted compression design value perpendicular to grain, MPa
$E_{x\ min}$	= modulus of elasticity of structural glued laminated timber for beam and column stability calculations for buckling about the x-x axis, MPa	$F_{c\perp x}$	= reference compression design value for bearing loads on the wide face of the laminations of structural glued laminated timber, MPa
E_y	= modulus of elasticity of structural glued laminated timber for deflections due to bending about the y-y axis, MPa	$F_{c\perp y}$	= reference compression design value for bearing loads on the narrow edges of the laminations of structural glued laminated timber, MPa
$E_{y\ min}$	= modulus of elasticity of structural glued laminated timber for beam and column stability calculations for buckling about the y-y axis, MPa	F_{rc}, F_{rc}'	= reference and adjusted radial compression design value for curved structural glued laminated timber members, MPa
F_b, F_b'	= reference and adjusted bending design value, MPa	F_{rt}, F_{rt}'	= reference and adjusted radial tension design value perpendicular to grain for structural glued laminated timber, MPa
F_b^*	= reference bending design value multiplied by all applicable adjustment factors except C_L , MPa	F_t, F_t'	= reference and adjusted tension design value parallel to grain, MPa
F_b^{**}	= reference bending design value multiplied by all applicable adjustment factors except C_V , MPa	F_v, F_v'	= reference and adjusted shear design value parallel to grain (horizontal shear), MPa
F_{b1}'	= adjusted edgewise bending design value, MPa	F_{vx}, F_{vx}'	= reference and adjusted shear design value for structural glued laminated timber members with loads causing bending about the x-x axis, MPa
F_{b2}'	= adjusted flatwise bending design value, MPa	F_{vy}	= reference shear design value for structural glued laminated timber members with loads causing bending about the y-y axis, MPa
F_{bE}	= critical buckling design value for bending members, MPa	F_{θ}'	= adjusted bearing design value at an angle to grain, MPa
F_{bx}^+	= reference bending design value for positive bending of structural glued laminated timbers, MPa	I	= moment of inertia, mm ⁴
F_{bx}^-	= reference bending design value for negative bending of structural glued laminated timbers, MPa	K_{cr}	= time dependent deformation (creep) factor
F_{by}	= reference bending design value of structural glued laminated timbers bent about the y-y axis, MPa	K_e	= buckling length coefficient for compression members
		K_{rs}	= empirical radial stress factor for double-tapered curved structural glued laminated timber bending members

K_ϕ	= empirical bending stress shape factor for double-tapered curved structural glued laminated timber	d_e	= depth of double-tapered curved structural glued laminated timber bending member at ends, mm
L	= span length of bending member, mm	d_e	= depth at the small end of a tapered straight structural glued laminated timber bending member, mm
M	= maximum bending moment, N-m	d_{equiv}	= depth of an equivalent prismatic structural glued laminated timber member, mm
N, N'	= reference and adjusted lateral design value at an angle to grain for a single bolt or a single split ring connector or shear plate connector unit, N	d_{max}	= the maximum dimension for that face of a tapered column, mm
P, P'	= reference and adjusted lateral design value parallel to grain for a single bolt or a single split ring connector or shear plate connector unit, N	d_{min}	= the minimum dimension for that face of a tapered column, mm
Q, Q'	= reference and adjusted lateral design value perpendicular to grain for a single bolt or a single split ring connector or shear plate connector unit, N	d_n	= depth of member remaining at a notch measured perpendicular to the length of the member, mm
Q	= statical moment of an area about the neutral axis, mm ³	d_{rep}	= representative dimension for tapered column, mm
R	= radius of curvature of inside face of structural glued laminated timber member, mm	d_y	= depth of structural glued laminated timber parallel to the wide face of the laminations when loaded in bending about the y-axis, mm
R_B	= slenderness ratio of bending member	d_1, d_2	= cross-sectional dimensions of rectangular compression member in planes of lateral support, mm
R_m	= radius of curvature at center line of structural glued laminated timber member, mm	e	= the distance the notch extends from the inner edge of the support, mm
S	= section modulus, mm ³	f_b	= actual bending stress, MPa
T	= Temperature, °C	f_c	= actual compression stress parallel to grain, MPa
V	= shear force, N	$f_{c\perp}$	= actual compression stress perpendicular to grain, MPa
V_r, V_r'	= reference and adjusted design shear, N	f_r	= actual radial stress in curved bending member, MPa
a	= support condition factor for tapered columns	f_t	= actual tension stress parallel to grain, MPa
b	= breadth (thickness) of rectangular bending member, mm	f_v	= actual shear stress parallel to grain, MPa
c	= distance from neutral axis to extreme fiber, mm	h	= vertical distance from the end of the double-tapered curved structural glued laminated timber beam to mid-span, mm
c	= constant for column stability factor	h_a	= vertical distance from the top of the double-tapered curved structural glued laminated timber supports to the beam apex, mm
d	= depth (width) of bending member, mm	ℓ	= span length of bending member, mm
d	= least dimension of rectangular compression member, mm	ℓ	= distance between points of lateral support of compression member, mm
d_c	= depth at peaked section of double-tapered curved structural glued laminated timber bending member, mm		
d_e	= effective depth of member at a connection, mm		

ℓ_b	= bearing length, mm
ℓ_c	= clear span, mm
ℓ_c	= length between tangent points for double-tapered curved structural glued laminated timber members, mm
ℓ_e	= effective span length of bending member, mm
ℓ_e	= effective length of compression member, mm
ℓ_{e1}, ℓ_{e2}	= effective length of compression member in planes of lateral support, mm
ℓ_e/d	= slenderness ratio of compression member
ℓ_u	= laterally unsupported span length of bending member, mm
t	= thickness, mm
x	= distance from beam support face to load, mm
Δ_H	= horizontal deflection at supports of symmetrical double-tapered curved structural glued laminated timber members, mm
Δ_{LT}	= immediate deflection due to the long-term component of the design load, mm
Δ_{ST}	= deflection due to the short term or normal component of the design load, mm
Δ_T	= total deflection from long-term and short-term loading, mm
Δ_c	= vertical deflection at mid-span of double-tapered curved structural glued laminated timber members, mm
θ	= angle of taper on the compression or tension face of structural glued laminated timber members, degrees
θ	= angle between the direction of load and the direction of grain (longitudinal axis of member) for split ring or shear plate connector design, degrees
ϕ_B	= angle of soffit slope at the ends of double-tapered curved structural glued laminated timber member, degrees
ϕ_T	= angle of roof slope of double-tapered curved structural glued laminated timber member, degrees
ω	= uniformly distributed load, N/mm

602.2 Definitions

See Section 202.

SECTION 603

MINIMUM QUALITY

603.1 Quality and Identification

All lumber, wood structural panels, particleboard, structural glued-laminated lumber, end-jointed lumber, fiberboard sheathing (when used structurally), hardboard siding (when used structurally), piles and poles regulated by this chapter shall conform to the applicable standards or grading rules specified in this chapter and shall be so identified by the grade mark or a certificate of inspection issued by an approved agency.

603.2 Minimum Capacity or Grade

Minimum capacity of structural framing members may be established by performance tests. When the tests are not made, capacity shall be based on design values specified in Section 615 and design calculations specified in Sections 616 to 618.

Approved end-jointed lumber may be used interchangeably with solid-sawn members of the same species and grade. Such use shall include, but not be limited to, light-framing joists, planks and decking.

Wood structural panels shall be of grades specified in accordance with Philippine National Standards (PNS).

603.3 Timber Connectors and Fasteners

Safe loads and design practices for types of connectors and fasteners not mentioned or fully covered in Section 619, may be determined in a manner approved by the Building Official.

The number and size of nails connecting wood members shall be in accordance with the applicable provisions of Section 619. Other connections shall be fastened to provide equivalent strength. End and edge distances and nail penetrations shall be in accordance with the applicable provisions of Section 619.

Fasteners for pressure preservative-treated and fire retardant-treated wood shall be of hot-dipped zinc coated galvanized, stainless steel, silicon bronze or copper. Fasteners required to be corrosion resistant shall be either zinc-coated fasteners, aluminum alloy wire fasteners or stainless steel fasteners.

Connections depending on joist hangers or framing anchors, ties, and other mechanical fastenings not otherwise covered may be used where approved by the Building Official.

603.4 Fabrication, Installation and Manufacture

Fabrication, installation, and manufacture of wood elements shall be in accordance with the following guidelines:

603.4.1 General

Preparation, fabrication and installation of wood members and their fastenings shall conform to accepted engineering practices and to the requirements of this chapter. All members shall be framed, anchored, tied and braced to develop the strength and rigidity necessary for the purposes for which they are used.

603.4.2 Timber Connectors and Fasteners

The installation of timber connectors and fasteners shall be in accordance with the provisions set forth in Section 619.

603.4.3 Metal Plate-Connected Wood Trusses

Metal plate-connected wood trusses shall conform to the provisions of Section 621. Each manufacturer of trusses using metal plate connectors shall retain an approved agency having no financial interest in the plant being inspected to make nonscheduled inspections of truss fabrication, delivery, and operations. The inspection shall cover all phases of truss operation, including lumber storage, handling, cutting, fixtures, presses or rollers, fabrication, bundling and banding, handling and delivery.

603.4.4 Structural Glued-Laminated Timber

The manufacture and fabrication of structural glued-laminated timber shall be under the supervision of qualified personnel.

603.4.5 Dried Fire Retardant-Treated Wood

Fire retardant-treated wood shall have been dried, following treatment, to moisture content (MC) not exceeding:

- 19% - for solid sawn lumber up to 50 mm thick
- 15% - for plywood

603.4.6 Size of Structural Members

Sizes of lumber referred to in this chapter are nominal sizes. Computations to determine the required sizes of members shall be based on the net dimensions (actual size) and not the nominal sizes. The rough size lumber shall not be less than the nominal size and the reduction in face dimensions of dressed lumber shall not be more than 6 mm of the nominal size.

603.4.7 Shrinkage

Consideration shall be given in the design to the possible effect of cross-grain dimensional changes considered vertically which may occur in lumber fabricated in a given condition.

603.4.8 Rejection

The building official may deny permission for the use of a wood member where permissible grade characteristics or defects are present in such a combination that they affect detrimentally the serviceability of the member.

SECTION 604 DESIGN AND CONSTRUCTION REQUIREMENTS

604.1 General

The following design requirements apply.

604.1.1 All wood structures shall be designed and constructed in accordance with the requirements of Section 601 up to Section 613.

604.1.2 Wind and earthquake load-resisting systems for all engineered wood structures shall be designed and constructed in accordance with the requirements of Section 614.

User Note: Alternatively, lateral load-resisting systems for single family dwellings may be proportioned according to the provisions of NSCP Volume III - Housing.

604.1.3 The design and construction of wood structures using working stress design (WSD) methods shall be in accordance with Section 615 to Section 618.

604.1.4 The design and construction of conventional light-frame wood structures shall be in accordance with Section 620.

604.1.5 The design and installation of timber connectors and fasteners shall be in accordance with Section 619.

604.1.6 Metal plate-connected wood trusses shall conform to the provisions of Section 621.

PART I - REQUIREMENTS APPLICABLE TO ALL DESIGN PROCEDURES

SECTION 605 DECAY AND TERMITE PROTECTION

605.1 Preparation of Building Site

All stumps and roots shall be removed from the soil to a depth of at least 300 mm below the surface of the ground in the area to be occupied by the building.

All wood forms which have been used in placing concrete, if within the ground or between foundation sills and the ground, shall be removed before a building is occupied or used for any purpose. Before completion, loose or casual wood shall be removed from direct contact with the ground under the building.

605.2 Wood Support Embedded in Ground

Wood embedded in the ground or in direct contact with the earth and used for the support of permanent structures shall be treated wood unless continuously below the ground waterline or submerged in fresh water. Round or rectangular posts, poles and sawn timber columns supporting permanent structures which are embedded in concrete or masonry in direct contact with the earth or embedded in concrete or masonry exposed to the weather shall be treated wood. The wood shall be treated for ground contact.

605.3 Under-Floor Clearance

When wood joists or the bottom of wood structural floors without joists are located closer than 450 mm or wood girders are located closer than 300 mm to exposed ground in crawl spaces or unexcavated areas located within the periphery of the building foundation, the floor assembly including posts, girders, joists and subfloor, shall be approved wood of natural resistance to decay or treated wood.

When the above under-floor clearances are required, the under-floor area shall be accessible. Accessible under-floor areas shall be provided with a minimum 450 mm by 600 mm opening unobstructed by pipes, ducts and similar construction. All under-floor access openings shall be effectively screened or covered. Pipes, ducts and other construction shall not interfere with the accessibility to or within under-floor areas.

605.4 Plates, Sills and Sleepers

All foundation plates or sills and sleepers on a concrete or masonry slab, which is in direct contact with earth, and sills that rest on concrete or masonry foundations, shall be treated wood, all marked or branded by an approved agency. Foundation wood marked or branded by an approved agency may be used for sills in localities subject to moderate hazard, where termite damage is not frequent and when specifically approved by the building official. In localities where hazard of termite is slight, any species of wood permitted by this chapter may be used for sills when specifically approved by the building official.

605.5 Columns and Posts

Columns and posts located on concrete or masonry floors or decks exposed to the weather or to water splash or in basements and which support permanent structures shall be supported by concrete piers or metal pedestals projecting above floors unless approved wood of natural resistance to decay or treated wood is used. The pedestal shall project at least 200 mm above exposed earth or at least 25 mm above finish floor level of such floors.

Individual concrete or masonry piers shall project at least 200 mm above exposed ground unless the supported columns or posts are treated wood or of approved wood with natural resistance to decay.

605.6 Girders Entering Masonry or Concrete Walls

Ends of wood girder entering masonry or concrete walls shall be provided with a 15 mm air space on tops, sides and ends unless approved wood of natural resistance to decay or treated wood is used.

605.7 Under-Floor Ventilation

Under-floor areas shall be ventilated by an approved mechanical means or by openings in under-floor area walls. Such openings shall have a net area of not less than 0.07 m^2 for each 10 m^2 of under-floor area. Openings shall be located as close to corners as practical and shall provide cross ventilation. The required area of such openings shall be approximately equally distributed along the length of at least two opposite sides. They shall be covered with corrosion-resistant wire mesh with mesh openings of 6 mm dimension. Where moisture due to climate and groundwater conditions is not considered excessive, the Building Official may allow operable louvers and may allow the required net area of vent opening to be reduced to 10 percent of the above, provided the under-floor ground surface area is covered with an approved vapor barrier.

605.8 Wood and Earth Separation

Protection of wood against deterioration as set forth in the previous sections for specified applications is required. In addition, wood used in construction of permanent structures and located nearer than 150 mm to earth shall be treated wood or wood of natural resistance to decay. Where located on concrete slabs placed on earth, wood shall be treated wood or wood of natural resistance to decay. Where not subject to water splash or to exterior moisture and located on concrete having a minimum thickness of 75 mm with an impervious membrane installed between concrete and earth, the wood may be untreated and of any species.

Where planter boxes are installed adjacent to wood frame walls, a 50-mm air space shall at least be provided between the planter and the wall. Flashing shall be installed when the air space is less than 150 mm in width. Where flashing is used, provisions shall be made to permit circulation of the air in the air space. The wood frame shall be provided with an exterior wall covering conforming to the provisions of Section 609.

605.9 Wood Supporting Roofs and Floors

Wood structural members supporting concrete or masonry slabs which are permeable to moisture and are exposed to the weather shall be approved wood of natural resistance to decay or treated wood unless separated from such floors or roofs by an impervious moisture barrier.

605.10 Moisture Content of Treated Wood

When wood which has been pressure-treated with a water-born preservative is used in enclosed locations where drying in service cannot readily occur, such wood must have a moisture content of 19 percent or less before being covered with insulation, interior wall finish floor covering or other materials.

605.11 Retaining Walls

All wood used as permanent parts of retaining or crib walls shall be treated wood.

605.12 Weather Exposure

Those portions of glued-laminated timbers that form the structural supports of a building or other structure and which are exposed to weather and not properly protected by a roof or eave overhangs of similar covering, shall be pressure-treated with an approved preservative or be manufactured from wood of natural resistance to decay.

All wood structural panels, when designed to be exposed in outdoor application, shall be of exterior type, except as provided in Section 605.2.

In geographical areas where experience has demonstrated a specific need, approved wood of natural resistance to decay or treated wood shall be used for those structural components of buildings or similar permanent building appurtenances when such members are exposed to the weather and are without adequate protection provided by a roof, eave, overhang or other covering against moisture or water accumulation on the surface or at joints between members. Such members may include: horizontal members such as girders, joists and decking; or vertical members such as posts, poles and columns; or both horizontal and vertical members.

605.13 Water Splash

Where wood-frame walls and partitions are covered on the interior with plaster, tile or similar materials and are subject to water splash, the framing shall be protected with approved waterproofing.

SECTION 606 WOOD SUPPORTING MASONRY OR CONCRETE

606.1 Dead Load

Wood members shall not be used to permanently support dead load of any masonry or concrete except in cases listed below or allowed by relevant sections of NSCP Volume III - Housing.

Exceptions:

1. *Masonry or concrete non-structural floor or roof surfacing not more than 100 mm thick may be supported by wood members.*
2. *Any structure may rest upon wood piles constructed in accordance with the requirements of Chapter 3 on "Earthworks and Foundations"*
3. *Veneer of brick or concrete stone may be supported by approved treated wood foundations when the maximum height of veneer does not exceed 9.0 m above the foundations. Such veneer used as an interior wall finish may also be supported on wood floors which are designed to support the additional load, and be designed to limit the deflection and shrinkage to 1/600 of the span of the supporting members.*
4. *Wood may be used to support glass block masonry having an installed weight of 98 kg/m² or less. When glass block is supported on wood floors, the floors shall be designed to limit deflection and shrinkage to 1/600 of the span of the supporting members and the allowable stresses for the framing members shall be reduced in accordance with Section 615.3.4.*

606.2 Horizontal Force

Wood members shall not be used to resist horizontal forces contributed by masonry or concrete construction in buildings over one story in height except where allowed by provisions of Section 614.2 of this chapter.

SECTION 607 WALL FRAMING

The framing of exterior and interior walls shall be in accordance with provisions specified in Section 620 unless a specific design is furnished.

Wood studs, walls and bearing partitions shall not support more than two floors and a roof unless an analysis satisfactory to the building official shows that shrinkage of wood framing will not have adverse effect upon the structure nor any plumbing, electrical, mechanical systems nor other equipment installed therein due to the excessive shrinkage or differential movements caused by shrinkage. The analysis shall also show that the roof drainage system and the foregoing systems or equipment will not be adversely affected or, as an alternate, such systems shall be designed to accommodate the differential shrinkage or movements.

SECTION 608 FLOOR FRAMING

Wood-joisted floors shall be framed and constructed and anchored to supporting wood stud or masonry walls.

Fire block and draft stops shall be in accordance with the following provisions:

1. In combustible construction, fire blocks and draft regulators shall be installed to cut off all concealed draft openings (both vertical and horizontal) and shall form an effective barrier between floors, between a top story and a roof or attic space, and shall subdivide attic spaces, concealed roof spaces and floor-ceiling assemblies. The integrity of all fire and draft stops shall be maintained.
2. Fire blocks shall be provided in the following locations:

- a. In concealed spaces of stud walls and partitions, including furred spaces, at the ceiling and floor levels, and at 3 m intervals at both horizontal and vertical length of the wall.

Exception:

Fire blocks may be omitted at floor and ceiling levels when approved smoke-actuated fire dampers are installed at these levels.

- b. At all interconnections between concealed vertical and horizontal spaces such as those that occur at soffits, drop ceilings, and cove ceilings.
- c. In concealed spaces between stair stringers, at the top and bottom of the run, and between studs along and in line with the run of the stairs if the walls under the stairs are unfinished.
- d. In openings around vents, pipes, ducts, chimneys, fireplaces, and similar openings which afford a passage for the fire at ceiling and floor levels, with noncombustible materials.
- e. At opening between attics spaced and chimneys chases for factory-built chimneys.
- f. Where wood sleepers are used for lying wood flooring on masonry or concrete fire-resistive floors, the space between the floor slab and the underside of the wood flooring shall be filled with noncombustible material or fire blocked in such a manner that there will be no open spaces

under the flooring that will exceed 9.3 m² in area and such space shall be filled solidly under all permanent partitions so that there is no communication under the flooring between adjoining rooms.

3. Fire blocks shall consist of 50 mm nominal lumber or one thickness of 18 mm plywood with joints backed by 18 mm plywood or one thickness of 19 mm Type 2-M particleboard. Fire blocks may also be of gypsum board, cement fiber board, mineral fiber, glass fiber or other approved materials securely fastened in place. Loose-fill insulation materials shall not be used as a fire block unless specifically tested in the form and manner intended for use to demonstrate its ability to remain in place and to retard the spread of fire and hot gases. Walls having parallel or staggered studs for sound-transmission control shall have fire blocks of batts or blankets of mineral fiber or glass fiber or other approved non-rigid materials.

4. Draft stops shall be provided in the following locations:

- a. Floor-Ceiling Assemblies.

- a.1 Single-family dwellings. As recommended in NSCP Volume III - Housing or when there is usable space above and below the concealed space of a floor-ceiling assembly in a single-family dwelling, draft stops shall be installed so that the area of the concealed space does not exceed 90 m². Draft stops shall divide the concealed space into approximately equal areas.

- a.2 Two or more dwelling units and hotels. Draft stops shall be installed in floor-ceiling assemblies of building having more than one dwelling unit and in hotels. Such draft stops shall be in line with walls separating tenants from each other and separating tenants from other areas.

- a.3 Other uses. Draft stops shall be installed in floor-ceiling assemblies of buildings or portions of buildings used for other than dwelling or hotel occupancies so that the area of concealed space does not exceed 90 m² and so that the horizontal dimension between stops does not exceed 18.0 m.

Exception:

Where approved automatic sprinklers are installed within the concealed space, the area between draft stops may be 270 m² and the horizontal dimension may be 30 m.

b. Attics

b.1 Single-family dwellings. Refer to NSCP Volume III - Housing.

b.2 Two or more dwelling unit and hotels. Drafts stops shall be installed in the attics, mansards, overhangs, false fronts set out from walls and similar concealed spaces of buildings containing more than one dwelling unit and hotels. Such drafts stop shall be above and in line with walls separating tenants from each other and from other uses.

Exceptions:

Draft stops may be omitted along one of the corridor walls, provided draft stops at tenant separation walls extend to the remaining corridor draft stop.

Where approved sprinklers are installed, draft stopping may be as specified in the exception below.

b.3 Other uses. Draft stops shall be installed in attics, mansards, overhangs, false fronts set out from walls and similar concealed spaces of buildings having uses other than dwellings or hotels so that the area between draft stops does not exceed 270 m² and the greatest horizontal dimension does not exceed 18.0 m.

Exception:

Where approved automatic sprinklers are installed, the area between the draft stops may be 800 m² and the greatest horizontal dimension may be 30 m.

b.4 Draft stopping materials shall be not less than 12 mm gypsum board, 9 mm plywood, 9 mm Type M-2 particleboard or other approved materials adequately supported. Openings in the partitions shall be protected by self-closing doors with automatic latches constructed as required for the partitions.

SECTION 609

EXTERIOR WALL COVERINGS

609.1 General

Exterior wood studs, walls shall be covered on the outside with the materials and in the manner specified in this section or elsewhere in this chapter. Studs or sheathing shall be covered on the outside face with a weather-resistant barrier when required. Exterior wall coverings of the minimum thickness specified in this section are based upon a maximum stud spacing of 400 mm unless otherwise specified.

609.2 Siding

Solid wood siding shall have an average thickness of 9 mm unless placed over sheathing permitted by this chapter.

Siding patterns known as rustic, drop siding or shiplap shall have an average thickness in place of not less than 15 mm and shall have a minimum thickness measured not less than 9 mm. Bevel siding shall have a minimum thickness measured at the butt section of not less than 11 mm and a tip thickness of not less than 5 mm. Siding of lesser dimensions may be used, provided such wall covering is placed over sheathing which conforms to the provisions specified elsewhere in this chapter.

All weatherboarding or siding shall be securely nailed to each stud with not less than one nail, or to solid 25 mm nominal wood sheathing or 12 mm plywood sheathing or 13 mm particleboard sheathing with not less than one line of nails spaced not more than 600 mm on center in each piece of the weatherboarding or siding.

Wood board siding applied horizontally, diagonally or vertically shall be nailed to studs, nailing strips or blocking set maximum 600 mm on center. Fasteners shall be nails or screws with a penetration of not less than 40 mm into studs, studs and wood sheathing combined, or blocking. Distance between such fastenings shall not exceed 600 mm for horizontally or vertically applied sidings and 800 mm for diagonally applied sidings.

609.3 Plywood

Where plywood is used for covering the exterior of outside walls, it shall be of the exterior type not less than 9 mm thick. Plywood panel siding shall be installed in accordance with Table 609.3-1. Unless applied over 25 mm wood sheathing or 12 mm wood structural panel sheathing or 13 mm particleboard sheathing, joints shall occur over framing members and shall be protected with a continuous wood batten, approved caulking, flashing, vertical or horizontal shiplaps or joints shall be lapped horizontally or otherwise made waterproof.

609.4 Shingles or Shakes

Wood shingles or shakes may be used for exterior wall covering, provided the frame of the structure is covered with building paper. All shingles or shakes attached to sheathing other than wood sheathing shall be secured with approved corrosion-resistant fasteners or on furring strips attached to the studs. Wood shingles or shakes may be applied over fiberboard shingle backer and sheathing with annular grooved nails. The thickness of wood shingles or shakes between wood nailing boards shall not be less than 9 mm. Wood shingles or shakes or siding may be nailed directly to approved fiberboard nail-based sheathing not less than 13 mm nominal thickness with annular grooved nails.

The weather exposure of wood shingle or shake siding used on exterior walls shall not exceed maximum set forth in Table 609.4-1.

609.5 Particleboard

When particleboard is used for covering the exterior of outside walls, it shall be of the M-1, M-S and M-2 Exterior Glue grades. Particleboard panel siding shall be installed in accordance with Table 609.5-1. Panel shall be gapped 3 mm and nails shall be spaced not less than 9 mm from edges and ends of sheathing. Unless applied over 16 mm net wood sheathing or 13 mm plywood sheathing or 13 mm particleboard sheathing, joints shall occur over framing members and shall be covered with a continuous wood batt; or joints shall be lapped horizontally or otherwise made waterproof to the satisfaction of the building official. Particleboard shall be sealed and protected with exterior quality finishes.

609.6 Hardboard

When hardboard siding is used for covering the outside of exterior walls, it shall conform to Table 609.6-1. Lap siding shall be installed horizontally and applied to sheathed or unsheathed walls. Corner bracing shall be installed in conformance with Section 620.9.3. A weather-resistive barrier shall be installed under the lap siding.

Square-edged, non-grooved panels and shiplap grooved or non-grooved siding shall be applied vertically to sheathed or unsheathed walls. Siding that is grooved shall not be less than 6 mm thick in the groove.

Nail size and spacing shall follow Table 609.6-1 and shall penetrate framing 40 mm. Lap siding shall overlap 25 mm minimum and be nailed through both courses and into framing members with nails located 12 mm from bottom of the overlapped course. Square-edged, non-grooved panels shall be nailed 9 mm from the perimeter of the panel and intermediately into studs. Shiplap edge panel siding with 9 mm shiplap shall be nailed 9 mm from the edges on both sides of the shiplap. The 19 mm shiplap shall be nailed 9 mm from the edge and penetrate through both the overlap and underlap. Top and bottom edges of the panel shall be nailed 9 mm from the edge. Shiplap and lap siding shall not be force fit. Square-edged panels shall maintain a 3-mm gap at joints. All joints and edges of siding shall be over framing members, and shall be made resistant to weather penetration with battens, horizontal overlaps or shiplaps to the satisfaction of the building official. A 3-mm gap shall be provided around all openings.

609.7 Nailing

All fasteners used for the attachment of siding shall be of a corrosion-resistant type.

Table 609.3-1 Exposed Plywood Panel Siding

Minimum Thickness ¹	Minimum Number of Plies	Stud Spacing
		Plywood Siding Applied Directly to Studs or Over Sheathing
10 mm	3	400 ²
12 mm	4	600

¹ Thickness of grooved panels is measured at bottom of grooves.

² 600 mm may be used if plywood siding applied with face grain perpendicular to studs or over one of the following:

(1) 25 mm board sheathing, (2) 10 mm wood structural panel sheathing or (3) 10 mm wood structural panel sheathing with strength axis (which is the long direction of the panel unless otherwise marked) of sheathing perpendicular to studs.

Table 609.4-1 Wood Shingle and Shake Side Wall Exposures

Shingle or Shake Length and Type	Maximum Weather Exposures (mm)			
	Single-Coursing		Double-Coursing	
	No. 1	No. 2	No. 1	No. 2
400 mm shingles	180	180	300	250
450 mm shingles	210	210	350	275
600 mm shingles	290	290	400	350
450 mm resawn shakes	180	-	350	-
450 mm straight-split shakes	180	-	400	-
600 mm resawn shakes	290	-	500	-

Table 609.5-1 Allowable Spans for Exposed Particleboard Panel Siding

Grade	Stud Spacing	Minimum Thickness		
		Siding		Exterior Ceilings and Soffits
		Direct to Studs	Continuous Support	Direct to Supports
M-1	400	16 mm	10 mm	10 mm
M-S				
M-2 "Exterior Glue"	600	16 mm	10 mm	10 mm

Table 609.6-1 Hardboard Siding

SIDING	Minimal Framing		Nail Size ^{1, 2} (mm)	Nail Spacing	
	Nominal Thickness (mm)	(50mm x 100mm) Maximum Spacing		General	Bracing Panels ³
1. Lap Siding					
Direct to Studs	10	400 mm o.c.	65	400 mm o.c.	Not applicable
Over Sheathing	10	400 mm o.c.	75	400 mm o.c.	Not applicable
2. Square Edge Panel Siding					
Direct to Studs	10	600 mm o.c.	50	150 mm o.c. at edges; 300 mm o.c. at intermediate supports	100 mm o.c. at edges; 200 mm o.c. intermediate supports
Over Sheathing	10	600 mm o.c.	65	150 mm o.c. at edges; 300 mm o.c. at intermediate supports	100 mm o.c. at edges; 200 mm o.c. intermediate supports
3. Shiplap Edge Panel Siding					
Direct to Studs	10	400 mm o.c.	50	150 mm o.c. at edges; 300 mm o.c. at intermediate supports	100 mm o.c. at edges; 200 mm o.c. intermediate supports
Over Sheathing	10	400 mm o.c.	65	150 mm o.c. at edges; 300 mm o.c. at intermediate supports	100 mm o.c. at edges; 200 mm o.c. intermediate supports

¹ Nails shall be corrosion resistant in accordance with Section 603.3.

² Minimum acceptable nail dimensions (mm).

	Panel Siding (mm)	Lap Siding (mm)
Shank diameter	2.5	2.5
Head diameter	6.0	6.0

³ When used to comply with Section 620.9.3.

SECTION 610 INTERIOR PANELING

All softwood wood structural panels shall conform to the provisions of the interior finishes and shall be installed in accordance with Table 619.3-1. Panels shall comply with UBC Standard 23-3 or its equivalent Philippine National Standards (PNS).

SECTION 611 SHEATHING

611.1 Structural Floor Sheathing

Structural floor sheathing shall be designed in accordance with the general provisions of this chapter and the special provisions in this section.

Sheathing used as subflooring shall be designed to support all loads specified in this chapter and shall be capable of supporting concentrated loads of not less than 1.33 kN without failure. The concentrated load shall be applied by a loaded disc, 75 mm or smaller in diameter.

Flooring, including the finish floor, underlayment and subfloor, where used, shall meet the following requirements:

1. Deflection under uniform design load limited to $1/360$ of the span between supporting joists or beams.
2. Deflection of flooring relative to joists under a 25 mm diameter concentrated load of 0.90 kN limited to 3 mm or less when loaded midway between supporting joists or beams not over 600 mm on center and $1/360$ of the span for spans over 600 mm.

Floor sheathing conforming to the provisions of Tables 611.1-1, 611.1-2, 611.1-3, or 611.1-4 shall be deemed to meet the requirements of this Section.

611.2 Structural Roof Sheathing

Structural roof sheathing shall be designed in accordance with the general provisions of this chapter and the special provisions in this section. Structural roof sheathing shall be designed to support all loads specified in this code and shall be capable of supporting concentrated loads of not less than 1.33 kN without failure. The concentrated load shall be applied by a loaded disk, 75 mm or smaller in diameter.

Deflection of structural roof sheathing under uniform design live and dead load shall be limited to $1/180$ of the span between supporting rafters or beams and $1/240$ under live load only.

Roof sheathing conforming to the provisions of Tables 611.1-1 or 611.1-2 and 611.1-3 shall be deemed to meet the requirements of this Section.

Wood structural panel roof sheathing shall be bonded by intermediate or exterior glue. Wood structural panel roof sheathing exposed on the underside shall be bonded with exterior glue.

Table 611.1-1 Allowable Spans for Lumber Floor and Roof Sheathing ¹

Span	Minimum Net Thickness (mm) of Lumber Placed			
	Perpendicular to Supports		Diagonally to Supports	
	Surfaced Dry ²	Surfaced Unseasoned	Surfaced Dry ²	Surfaced Unseasoned
FLOOR				
600	20	20	20	20
400	16	18	16	18
ROOF				
600	16	18	20	20

¹ Floor or roof sheathing conforming to this table shall be deemed to meet the design criteria of Section 611.

² Maximum 19% moisture content.

Table 611.1-2 Allowable Spans and Loads for Wood Structural Panel Sheathing and Single-Floor Grades Continuous Over Two or More Spans with Strength Axis Perpendicular to Supports ^{1,2}

Sheathing Grades		ROOF ³				FLOOR ⁴
Panel Span Rating	Panel Thickness (mm)	Maximum Span (mm)		Load ⁵ (kN/m ²)		Maximum Span (mm)
Roof/Floor Span (mm/mm)		With Edge Support ⁶	Without Edge Support	Total Load	Live Load	
300/0	8	300	300	1.92	1.44	0
400/0	8, 9	400	400	1.92	1.44	0
500/0	8, 9	500	500	1.92	1.44	0
600/0	10, 12	600	500 ⁷	1.92	1.44	0
600/400	12	600	600	2.40	1.92	400
800/400	12, 16	800	700	1.92	1.44	400 ⁸
1000/500	16, 20, 22	1000	800	1.92	1.44	500 ^{8,9}
1200/600	20, 22	1200	900	2.16	1.68	600
1350/800	22, 25	1350	1000	2.16	1.68	800
1500/1200	22, 25, 30	1500	1200	2.16	1.68	1200
Single-Floor Grades		ROOF ³				FLOOR ⁴
Panel Span Rating (mm)	Panel Thickness (mm)	Maximum Span (mm)		Load ⁵ (kN/m ²)		Maximum Span (mm)
Roof/Floor Span		With Edge Support ⁶	Without Edge Support	Total Load	Live Load	
400 o.c.	12, 16	600	600	2.40	1.92	400
500 o.c.	16, 20	800	800	1.92	1.44	500
600 o.c.	20	1200	900	1.68	1.20	600
800 o.c.	22	1200	1000	2.40	1.92	800
1200 o.c.	28, 30	1500	1200	2.40	2.40	1200

¹ Applies to panels 600 mm or wider.

² Floor and roof sheathing conforming to this table shall be deemed to meet the design criteria of Section 611.

³ Uniform load deflection limitations 1/180 of span under live load plus dead load, 1/240 under live load only.

⁴ Panel edges shall have approved tongue-and-groove joints or shall be supported with blocking unless 6 mm minimum thickness underlayment or 40 mm of approved cellular or lightweight concrete is placed over the subfloor, or finish floor is 20 mm wood strip. Allowable uniform load based on deflection of 1/360 of span is 4.8 kN/m². Note: the span rating of 1200 mm on center is based on a total load of 3.1 kN/m².

⁵ Allowable load at maximum span.

⁶ Tongue-and-groove edges, panel edge clips (one midway between each support, except two equally spaced between supports 1200 mm on center), lumber blocking, or other. Only lumber blocking shall satisfy blocked diaphragms requirements.

⁷ For 12 mm panel, maximum span shall be 600 mm.

⁸ May be 600 mm on center where 20 mm wood strip flooring is installed at right angles to joist.

⁹ May be 600 mm on center for floors where 40 mm of cellular or lightweight concrete is applied over the panels.

Table 611.1-3 Allowable Loads for Wood Structural Panel Roof Sheathing Continuous Over Two or More Spans and Strength Axis Parallel to Supports (Plywood structural panels are five-ply, five-layer unless otherwise noted)^{1, 2}

Panel Grade	Thickness (mm)	Maximum Span (mm)	Load at Maximum Span (kN/m ²)	
			Live	Total
Structural 1	12	600	0.96	1.44
	12	600	1.68 ³	2.16 ³
	12	600	1.92 ³	2.40 ³
	16	600	3.35	3.83
	20	600	4.31	4.79
Other Grades Covered in UBC Standard 23-2 or 23-3	12	400	1.92	2.40
	12	600	0.96	1.20
	12	600	1.20	1.44
	16	600	1.92 ³	2.40 ³
	16	600	2.16 ³	2.63 ³
	20	600	2.87 ³	3.11 ³

¹ Roof sheathing conforming to this table shall be deemed to meet the design criteria of Section 611.

² Uniform load deflection limitations: 1/180 of span under live load plus dead load, 1/240 under live load only. Edges shall be blocked with lumber or other approved type of edge supports.

³ For composite and four-ply plywood structural panel, load shall be reduced by 0.72 kN/m².

Table 611.1-4 Allowable Span for Wood Structural Panel Combination Subfloor-Underfloor-Underlayment (Single Floor)^{1, 2}
Panels Continuous over Two or More Spans and Strength Axis Perpendicular to Supports

Identification	Maximum Spacing of Joists (mm)				
	400	500	600	800	1200
Species Group ³	Thickness (mm)				
I	12	16	20	-	-
II, III	16	20	22	-	-
IV	20	22	25	-	-
Span rating ⁴	400 o.c.	500 o.c.	600 o.c.	800 o.c.	1200 o.c.

¹ Spans limited to value shown because of possible effects of concentrated loads. Allowable uniform loads based on deflection of 1/360 of span is 4.8 kN/m². Note: the allowable total uniform load for 30 mm wood structural panels over joists spaced 1200 mm on center is 3.1 kN/m². Panel edges shall have approved tongue-and-groove joints or shall be supported with blocking, unless 6 mm minimum thickness underlayment or 38 mm of approved cellular or lightweight concrete is placed over the subfloor, or finish floor is 20 mm wood strip.

² Floor panels conforming to this table shall be deemed to meet the design criteria of Section 611.

³ Applicable to all grades of sanded exterior-type plywood. See UBC Standard 23-2 for plywood species groups.

⁴ Applicable to underlayment grade and C-C (plugged) plywood, and single floor grade wood structural panels.

Table 611.1-5 Allowable Spans for Particleboard Subfloor and Combined Subfloor-Underlayment ^{1,2}

Grade	Thickness (mm)	Maximum Spacing of Supports (mm) ³	
		Subfloor	Combined Subfloor-Underlayment ^{4, 5}
2-M-W	12	400	-
	16	500	400
	20	600	600
2-M-3	20	500	500

¹ All panels are continuous over two or more spans.

² Floor sheathing conforming to this table shall be deemed to meet the design criteria of Section 611.

³ Uniform deflection limitation: $1/360$ of the span under 4.8 kN/m^2 minimum load.

⁴ Edges shall have tongue-and-groove joints or shall be supported with blocking. The tongue-and-groove panels are installed with the long dimension perpendicular to supports.

⁵ A finish wearing surface is to be applied to the top of the panel.

SECTION 612 MECHANICALLY-LAMINATED FLOORS AND DECKS

A laminated lumber floor or deck built up of wood members set on edge, when meeting the following requirements, may be designed as a solid floor or roof deck of the same thickness and continuous span may be designed on the basis of the full cross section using the simple span moment coefficient.

Nail length shall not be less than $2\frac{1}{2}$ times the net thickness of each lamination. When deck supports are 1.20 m on center or less, side nails shall be spaced not more than 750 mm on center and staggered one-third of the spacing in adjacent laminations. When supports are spaced more than 1.20 m on center, side nails shall be spaced not more than 450 mm on center alternately near top and bottom edges, and also staggered one-third of the spacing in adjacent laminations. Two side nails shall be used at each end of butt-jointed pieces.

Laminations shall be toe-nailed to supports with 100 mm or larger common nails. When supports are 1.20 m on center or less, alternate laminations shall be toe-nailed to alternate supports; when supports are spaced more than 1.20 m on center, alternate laminations shall be toe-nailed to every support.

A single-span deck shall have all laminations full length.

A continuous deck of two spans shall not have more than one splice every fourth lamination and placed within quarter spans of adjoining supports.

Joints shall be closely butted over supports or staggered across the deck but within the adjoining quarter spans.

No lamination shall be spliced more than twice in any span.

SECTION 613 POST-BEAM CONNECTIONS

Where post and beam or girder construction is used, the design shall be in accordance with the provisions of this chapter. Positive connection shall be provided to ensure against uplift and lateral displacement.

PART II - REQUIREMENTS APPLICABLE TO DESIGN OF WIND AND EARTHQUAKE LOAD- RESISTING SYSTEMS

SECTION 614 WOOD SHEAR WALLS AND DIAPHRAGMS

614.1 General

Unless permitted by the Building Official or by relevant provisions of NSCP Volume III - Housing, use of wood shear walls and diaphragms shall be limited to 1 to 2-story dwellings. Where applicable, succeeding provisions of this Section shall be used as bases for their design.

Particleboard vertical diaphragms and lumber and wood structural panel horizontal and vertical diaphragms may be used to resist horizontal forces in horizontal and vertical distributing or resisting elements, provided the deflection in the plane of the diaphragms, as determined by calculations, tests or analogies drawn therefrom, does not exceed the permissible deflection of attached distributing or resisting elements.

Permissible deflection shall be that deflection up to which a diaphragm and any attached distributing or resisting element will maintain its structural integrity under assumed load conditions, i.e. continue to support assumed loads without danger to occupant of the structure.

Connections and anchorages capable of resisting the design forces shall be provided between the diaphragms and the resisting elements. Openings in diaphragms which materially affect their strength shall be fully detailed on the plans and shall have their edges adequately reinforced to transfer all shearing stresses.

Size and shape of each horizontal diaphragm and shear wall shall be limited as set forth in Table 614.1-1. The height of a shear wall shall be defined as:

1. The maximum clear height from foundation to bottom of diaphragm framing above, or
2. The maximum clear height from top of diaphragm to bottom of diaphragm framing above.

The width of a shear wall shall be defined as the width of sheathing.

Where shear walls with openings are designed for force transfer around the openings, the limitations of Table 614.1-1 shall apply to the overall shear wall including openings and to each wall pier at the side of an opening. The height of a wall pier shall be defined as the clear height of the pier at the side of an opening. The width of a wall pier shall be defined as the sheathed width of the pier at the side of an opening. Design for force transfer shall be based on a rational analysis.

Table 614.1-1 Maximum Diaphragm Dimension Ratios

Material	Horizontal Diaphragms	Vertical Diaphragms
	Maximum Span-Width Ratios	Maximum Height-Width Ratios
1. Diagonal sheathing, conventional	3:1	1:1 ¹
2. Diagonal sheathing, special	4:1	2:1 ²
3. Wood structural panels and particleboard, nailed all edges	4:1	2:1 ²
4. Wood structural panels and particleboard, blocking omitted at intermediate joints.	4:1	³

¹ In Seismic Zone 2, the maximum ratio may be 2:1.

² In Seismic Zone 2, the maximum ratio may be 3½:1.

³ Not permitted.

In buildings of wood-frame construction where rotation is provided for, the depth of the diaphragm normal to the open side shall not exceed 7.50 m or two-thirds the diaphragm width, whichever is the smaller depth. Straight sheathing shall not be permitted to resist shears in diaphragms acting in rotation.

Exceptions:

1. One-story, wood-framed structures with the depth normal to the open side not greater than 7.50 m. may have a depth equal to the width.
2. Where calculations show that diaphragm deflections can be tolerated, the depth normal to the open end may be increased to a depth-to-width ratio not greater than 1½:1 for diagonal sheathing or 2:1 for special diagonal sheathed or plywood or particleboard diaphragms.

In masonry or concrete buildings, lumber and wood structural panel diaphragms shall not be considered as transmitting lateral forces by rotation.

Diaphragm sheathing nails or other approved sheathing connectors shall be driven flush but shall not fracture the surface of the sheathing.

614.2 Wood Members Resisting Horizontal Forces Contributed by Masonry and Concrete

Wood members shall not be used to resist horizontal forces contributed by masonry or concrete construction in buildings over one story in height.

Exceptions:

1. *Wood floor and roof members may be used in horizontal trusses and diaphragms to resist horizontal forces imposed by wind, earthquake or earth pressure, provided such forces are not resisted by rotation of the truss or diaphragm.*
2. *Vertical wood structural panel-sheathed shear walls may be used to provide resistance to wind or earthquake forces in two-story buildings of masonry or concrete construction, provided the following requirements are met*
 - a. *Story-to-story wall heights shall not exceed 3.6 m.*
 - b. *Horizontal diaphragm shall not be considered to transmit lateral forces by rotation or cantilever action.*
 - c. *Deflection of horizontal and vertical diaphragms shall not permit per-story deflection of supported masonry or concrete walls to exceed 0.0025 times each story height.*
 - d. *Wood structural panel sheathing in horizontal diaphragms shall have all unsupported edges blocked. Wood structural panel sheathing for both stories of vertical diaphragms shall have all unsupported edges blocked and for the lower-story walls shall have a minimum thickness of 12 mm.*
 - e. *There shall be no out-of-plane horizontal offsets between the first and second stories of wood structural panel shear walls.*

614.3 Wood Diaphragms

Wood diaphragms shall conform to the following guidelines:

614.3.1 Conventional Lumber Diaphragm Construction

Such lumber diaphragms shall be made up of 25 mm nominal sheathing boards laid at an angle of approximately 45 degrees to supports. Sheathing boards shall be directly nailed to each intermediate bearing member with not less than two 65-mm nails for 25 mm by 150 mm nominal boards and three 65-mm nails for boards 200 mm or wider; and in addition, three 65-mm nails and four 65-mm nails shall be used for 150 mm and 200 mm boards, respectively, at the diaphragm boundaries. End joints in adjacent boards shall be separated by at least one joist or stud space, and there shall be at least two boards between joints on the same support. Boundary members at edges of diaphragms shall be designed to resist direct tensile or compressive chord stresses and adequately tied together at corners.

614.3.2 Special Lumber Diaphragm Construction

Special diagonally sheathed diaphragms shall conform to conventional construction and in addition, shall have all elements designed in conformance with the provisions of this chapter.

Each chord or portion thereof maybe considered as a beam loaded with a uniform load per meter equal to 50 percent of the unit shear due to diaphragm action. The load shall be assumed as acting normal to the chord, in the plane of the diaphragm and either towards or away from the diaphragm.

The span of chord, or portion thereof, shall be the distance between structural members of the diaphragm such as the joists, studs and blocking, which serve to transfer the assumed load to the sheathing.

Special diagonally sheathed diaphragms shall include conventional diaphragms sheathed with two layers of diagonal sheathing at 90 degrees to each other and on the same face of the supporting members.

614.3.3 Wood Structural Panel Diaphragm

Horizontal and vertical diaphragms sheathed with wood structural panels may be used to resist horizontal forces for horizontal diaphragm and for vertical diaphragms, or may be calculated by principles of mechanics without limitation by using values of nail strength and wood structural panel shear values as specified elsewhere in this chapter. Wood structural panels for horizontal diaphragms shall be as set forth in Tables 611.1-2 and 611.1-3 for corresponding joist spacing and loads. Wood structural panels in shear walls shall be at least 8 mm thick for studs spaced 400 mm on center and 9 mm thick where studs are spaced 600 mm on center.

Maximum spans for wood structural panel subfloor underlayment shall be as set forth in Table 611.1-4. Wood structural panels used for horizontal and vertical diaphragms shall conform to UBC Standard 23-2 and UBC Standard 23-3 or equivalent PNS.

All boundary members shall be proportioned and spliced where necessary to transmit direct stresses. Framing members shall be at least 50-mm nominal in the dimension to which the plywood is attached. In general, panel edges shall bear on the framing members and butt along their centerlines. Nails shall be placed not less than 10 mm from the panel edge, shall be spaced not more than 150 mm on center along panel edge bearings, and shall be firmly driven into the framing members. No unblocked panels less than 300 mm wide shall be used.

Diaphragms with panel edges supported in accordance with Tables 611.1-2, 611.1-3 and 611.1-4 shall not be considered as blocked diaphragms unless blocking or other means of shear transfer is provided.

614.4 Particleboard Diaphragms

Vertical diaphragms sheathed with particleboard may be used to resist horizontal forces.

All boundary members shall be proportioned and spliced where necessary to transmit direct stresses. Framing members shall be at least 50-mm nominal in the dimension to which the particleboard is attached. In general, panel edges shall bear on the framing members and butt along their centerlines. Nails shall be placed not less than 10 mm from the panel edge, shall be spaced not more than 150 mm on center along panel edge bearings, and shall be firmly driven into the framing members. Unblocked panels less than 300 mm wide shall not be allowed or used.

Diaphragms with panel edges supported in accordance with Table 611.1-4 shall not be considered as blocked diaphragms unless blocking or other means of shear transfer is provided.

614.5 Wood Shear Walls and Diaphragms in Seismic Zone 4

Section 614.5.1 to 614.5.5 shall be used for wooden shear walls and diaphragms design in Seismic Zone 4 areas.

614.5.1 Scope

Design and construction of wood shear walls and diaphragms in Seismic Zone 4, as allowed by provisions of Section 614.1 and NSCP Volume III - Housing, shall conform to the requirements of this section.

614.5.2 Framing

Collector members shall be provided to transmit tension and compression forces. Perimeter members at openings shall be provided and shall be detailed to distribute the shearing stresses. Diaphragm sheathing shall not be used to splice these members.

Diaphragm chords and ties shall be placed in, or tangent to, the plane of the diaphragm framing unless it can be demonstrated that the moments, shear and deflections and deformations resulting from other arrangements can be tolerated.

614.5.3 Wood Structural Panel

Wood structural panels shall be manufactured using exterior glue.

Wood structural panel diaphragms and shear walls shall be constructed with wood structural panel sheets not less than 1.20 m by 2.40 m, except at boundaries and changes in framing where minimum sheet dimension shall be 600 mm unless all edges of the undersized sheets are supported by framing members or blocking.

Framing members or blocking shall be provided at the edges of all sheets in shear walls.

Wood structural panel sheathing may be used for splicing members, other than those noted in Section 614.5.2, where the additional nailing required to develop the transfer of forces will not cause cross-grain bending or cross-grain tension in the nailed member.

614.5.4 Heavy Wood Panels

Diagonally sheathed panels utilizing 50-mm nominal boards may be used to resist the same permissible shear as 25-mm nominal lumber, except that 90 mm nails shall be used instead of 65 mm.

Panels utilizing straight decking overlaid with plywood may be used to resist shear forces using the same shear values as permitted for the wood structural panel alone. Wood structural panel joints parallel to the decking shall be located at least 25 mm offset from any parallel decking joint.

Heavy decking panels utilizing dowel pins, or vertically laminated panels connected by nailing units to one another, resist shear forces based on the permissible shear values of their connectors.

614.5.5 Particleboard

Particleboard shall not be less than Type M “Exterior Glue”.

Shear walls shall be sheathed with particleboard sheets not less than 1.20 m by 2.40 m except at boundaries and changes in framing. The required nail size and spacing in Table 619.3-1 apply to panel edges only. All panel edges

shall be backed with 50 mm nominal or wider framing. Sheets are permitted to be installed either horizontally or vertically. For 10-mm particleboard sheets installed with the long dimension parallel to the studs spaced 600 mm on center, nails shall be spaced at 150 mm on center along intermediate framing members. For all other conditions, nails of the same size shall be spaced at 300 mm on center along intermediate framing members.

614.6 Fiberboard Sheathing Diaphragms

Wood stud walls sheathed with fiberboard sheathing may be used to resist horizontal forces not exceeding those set forth in this chapter. The fiberboard sheathing, 1.2 m by 2.4 m, shall be applied vertically to wood studs not less than 50 mm nominal in thickness spaced 400 mm on center. Nailing shown in Table 614.6-1 shall be provided at the perimeter of the sheathing board and at the intermediate studs. Blocking of not less than 50-mm nominal thickness shall be provided at horizontal joints when wall height exceeds length of sheathing panel, and sheathing shall be fastened to the blocking with nails sized as shown in Table 616.6-1 spaced 75 mm on centers each side of joint. Nails shall be spaced not less than 10 mm from edges and ends of sheathing. Marginal studs of shear walls or shear-resisting elements shall be adequately anchored at the top and bottom and designed to resist all forces. The maximum height-width ratio shall be 1½:1.

Table 614.6-1 Allowable Shears for Wind or Seismic Loading on Vertical Diaphragms of Fiberboard Sheathing Board for Type V Construction Only¹

Size and Application	Nail Size	Shear Value in 75 mm Nail Spacing Around Perimeter and 150 mm at Intermediate Points, (N/mm)
12 x 1200 x 2400 mm	Galvanized roofing nail 40 mm long, 10 mm head	182 ²
20 x 1200 x 2400 mm	Galvanized roofing nail 45 mm long, 10 mm head	256

¹ Fiberboard sheathing diaphragms shall not be used to brace concrete or masonry walls.

² The shear value may be 780 N for 12 by 1200 by 2400 mm fiberboard nail-base sheathing.

SECTION 615

DESIGN VALUES FOR STRUCTURAL MEMBERS

615.1 General

615.1.1 General Requirement

Each wood structural member or connection shall be of sufficient size and capacity to carry the applied loads without exceeding the adjusted design values specified herein.

Calculation of adjusted design values shall be determined using the applicable adjustment factors specified herein.

615.1.2 Responsibility of Designer to Adjust for Conditions of Use

Adjusted design values for wood members and connections in particular end uses shall be appropriate for the conditions under which the wood is used, taking into account the differences in wood strength properties with different moisture contents, load durations, and types of treatment. Common end use conditions are addressed in this Specification. It shall be the final responsibility of the designer to relate design assumptions and reference design values, and to make design value adjustments appropriate to the end use.

615.2 Reference Design Values

Reference design values are set forth in Table 615.2-1. Stress grading may be based on Table 615.2-3. Design value adjustments for wood products are based on methods specified in each of the wood product sections. Sections 617 to 618 contain design provisions for sawn lumber and glued laminated timber. Section 619 contains design provisions for connections. Reference design values are for normal load duration under the moisture service conditions specified.

615.3 Adjustment of Reference Design Values

615.3.1 Applicability of Adjustment Factors

Reference design values shall be multiplied by all applicable adjustment factors to determine adjusted design values. The applicability of adjustment factors for sawn lumber and structural glued laminated timber is defined in Section 617 to 618.

615.3.2 Load Duration Factor, C_D

615.3.2.1 Wood has the property of carrying substantially greater maximum loads for short durations than for long durations of loading. Reference design values apply to normal load duration. Normal load duration represents a load that fully stresses a member to its allowable design value by the application of the full design load for a cumulative duration of approximately ten years. When the cumulative duration of the full maximum load does not exceed the specified time period, all reference design values except modulus of elasticity, E , modulus of elasticity for beam and column stability, E_{min} , and compression perpendicular to grain, $F_{c\perp}$, based on a deformation limit (see 617.2.6) shall be multiplied by the appropriate load duration factor, C_D , from Table 615.3-1 to take into account the change in strength of wood with changes in load duration.

615.3.2.2 The load duration factor, C_D , for the shortest duration load in a combination of loads shall apply for that load combination. All applicable load combinations shall be evaluated to determine the critical load combination. Design of structural members and connections shall be based on the critical load combination.

615.3.2.3 The load duration factors, C_D , in Table 615.3-1 are independent of load combination factors, and both shall be permitted to be used in design calculations.

Table 615.3-1 Load Duration Factors¹, C_D

Load Duration	C_D	Typical Design Loads
Permanent	0.9	Dead Load
Ten years	1.0	Occupancy Live Load
Seven days	1.25	Construction Load
One day	1.33	Earthquake Load
One day	1.33	Wind Load (Connections and Fasteners only)
Ten minutes	1.60	Wind Load (members)
Impact ²	2.0	Impact Load

¹When using the Alternative Basic Load Combinations of Chapter 2, the one-third increase shall not be used concurrently with the Load Duration Factor, C_D .

²Load duration factor greater than 1.6 shall not apply to structural members pressure-treated with water-borne preservatives or fire retardant chemicals. The impact load duration shall not apply to connections.

615.3.3 Temperature Factor, C_t

Reference design values shall be multiplied by the temperature factors, C_t , in Table 615.3-2 for structural members that will experience sustained exposure to elevated temperatures up to 66°C.

Table 615.3-2 Temperature Factor, C_t

Reference Design Values	In Service Moisture Conditions ¹	C_t		
		$T \leq 38^\circ\text{C}$	$T \leq 52^\circ$ $> 38^\circ$	$T \leq 66^\circ$ $> 52^\circ$
F_t, E	Wet or Dry	1.0	0.90	0.90
$F_b, F_v,$	Dry	1.0	0.80	0.70
F_c and $F_{c\perp}$	Wet	1.0	0.70	0.50

¹ Wet service condition for sawn lumber is defined as in service moisture content greater than 19%. Wet service condition for glued laminated timber is defined as in service moisture content of 16% or greater.

615.3.4 Fire Retardant treatment

The effects of fire retardant chemical treatment on strength shall be accounted for in the design. Adjusted design values, including adjusted connection design values, for lumber and structural glued laminated timber pressure-treated with fire retardant chemicals shall be obtained from the company providing the treatment and redrying service. Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with fire retardant chemicals. (See Table 615.3-1)

Table 615.2-1 Reference Values for Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods¹

Species (Common and Botanical Names)	80% Stress Grade				
	Bending and Tension Parallel to Grain	Modulus of Elasticity in Bending	Compression Parallel to Grain	Compression Perpendicular to Grain	Shear Parallel to Grain
	F_b, F_t	E	F_c	$F_{c\perp}$	F_v
	MPa	$\times 10^4$ MPa	MPa	MPa	MPa
I. High Strength Group					
Agoho (<i>Casuarina equisetifolia</i> Forst.)	26.3	8.22	14.5	5.91	2.95
Liusin [<i>Parinari corymbosa</i> (Blume) Miq.]	25.0	9.36	15.6	4.31	2.64
Malabayabas (<i>Tristania</i> spp.)	28.7	8.30	15.8	8.70	3.02
Manggachapui (<i>Hopea</i> spp.)	25.8	9.63	16.0	6.03	2.78
Molave (<i>Vitex parviflora</i> Juss.)	24.0	6.54	15.4	6.34	2.88
Narig (<i>Vatica</i> spp.)	21.8	8.33	13.7	4.97	2.61
Sasalit [<i>Tejmanniodendron ahernianum</i> (Merr) Bkh.]	31.3	9.72	21.60	10.2	3.38
Yakal (<i>Shorea</i> spp.)	24.5	9.78	15.8	6.27	2.49
II. Moderately High Strength Group					
Antipolo (<i>Arthocarpus</i> spp.)	18.6	5.35	10.8	3.90	2.06
Binggas (<i>Terminalia</i> spp.)	18.9	6.57	11.4	3.27	2.24
Bokbok [<i>Xanthophyllum excelsum</i> (Blume) Miq.]	18.1	6.36	11.3	3.41	2.18
Dao (<i>Dracontomelon</i> spp.)	16.2	5.43	9.44	2.27	1.92
Gatasan [<i>Garcinia venulosa</i> (Blanco) Choisy]	20.8	6.84	13.5	3.52	2.36
Guijo (<i>Shorea</i> spp.)	21.8	8.47	13.2	4.26	2.40
Kamagong (<i>Diospyros</i> spp.)	20.9	7.20	11.7	4.39	2.47
Kamatog [<i>Erythrophloeum densiflorum</i> (Elm) Merr.]	19.0	7.56	11.2	3.95	2.35
Katmon (<i>Dillenia</i> spp.)	18.8	6.82	11.9	4.84	2.29
Kato (<i>Amoora</i> spp.)	18.4	8.04	10.6	3.46	1.96
Lomarau (<i>Swintonia foxworthyi</i> Elm.)	19.8	7.92	11.8	2.98	2.18
Mahogany, Big-leafed (<i>Swietenia macrophylla</i> King)	16.5	4.66	10.5	3.83	2.71
Makaasim (<i>Syzygium nitidum</i> Benth)	20.5	6.72	11.4	3.70	2.40
Malakauayan [<i>Decosocarpus philippinensis</i> (Foxw.) de Laub.]	18.9	6.66	11.12	2.32	2.14
Narra (<i>Pterocarpus indicus</i> Willd)	18.0	5.94	11.4	3.07	1.91
Pahunan (<i>Mangifera</i> spp.)	16.6	6.53	10.0	2.50	2.05
III. Medium Strength Group					
Apitong (<i>Dipterocarpus</i> spp.)	16.5	7.31	9.56	2.20	1.73
Bagtikan [<i>Parashorea malaanonan</i> (Blanco) Merr.]	16.6	6.48	9.89	2.33	1.82
Dangkalan (<i>Calophyllum</i> spp.)	16.3	6.38	9.20	2.48	1.98
Gisau (<i>Canarium</i> spp.)	14.3	5.33	8.16	1.99	1.90
Lanut-an-bagyo [<i>Gonystylus macrophyllum</i> (miq.) Airy Shaw]	15.0	6.06	8.96	2.02	1.84
Lauan (<i>Shorea</i> spp.)	13.9	5.83	8.18	1.72	1.48
Malaanonang (<i>Shorea</i> spp.)	13.8	5.41	8.54	1.96	1.59
Malasaging (<i>Aglaiia</i> spp.)	16.8	5.94	9.51	2.92	1.85
Malugai (<i>Pometia</i> spp.)	15.4	6.30	9.33	3.07	2.07
Miau (<i>Dysoxylum</i> spp.)	15.7	6.50	8.83	2.78	2.06
Nato (<i>Palaquium</i> spp.)	16.2	5.56	9.17	2.33	1.98
Palosapis (<i>Anisoptera</i> spp.)	13.8	5.98	8.38	2.73	1.68
Pine (<i>Pinus</i> spp.)	14.7	6.66	8.29	1.88	1.56
Salakin (<i>Aphanamixis</i> spp.)	15.7	5.67	8.83	2.94	1.88
Vidal lanutan [<i>Hibiscus campylosiphon</i> Turcz. var. <i>glabrecens</i> (Har. Ex. Perk.)]	19.5	5.83	8.54	2.65	2.39
IV. Moderately Low Strength Group					
Almaciga [<i>Agathis dammara</i> (Lamb.) Rilh.]	11.8	5.47	6.27	1.44	1.47
Bayok (<i>Pterospermum</i> spp.)	12.6	4.75	7.33	1.30	1.20
Lingo-lingo (<i>Vitex turczaninowii</i> Merr.)	13.2	4.13	6.85	2.00	1.66
Mangasinoro (<i>Shorea</i> spp.)	12.8	5.36	7.46	1.97	1.44
Raintree [<i>Samanea saman</i> (Jacq.) Merr.]	11.9	2.75	7.23	3.32	2.07
Yemane (<i>Gmelina arborea</i> R. Br.)	12.6	4.09	7.87	3.40	1.96

¹ See Table 615.2-2 for Reference Values for Other Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods

Table 615.2-1 (Cont'd) Reference Values for Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods¹

Species (Common and Botanical Names)	63% Stress Grade				
	Bending and Tension Parallel to Grain	Modulus of Elasticity in Bending	Compression Parallel to Grain	Compression Perpendicular to Grain	Shear Parallel to Grain
	F_b, F_t	E	F_c	$F_{c\perp}$	F_v
	MPa	$\times 10^3$ MPa	MPa	MPa	MPa
I. High Strength Group					
Agoho (<i>Casuarina equisetifolia</i> Forst.)	20.7	6.47	11.4	4.65	2.32
Liusin [<i>Parinari corymbosa</i> (Blume) Miq.]	19.7	7.37	12.3	3.39	2.08
Malabayabas (<i>Tristania</i> spp.)	22.6	6.53	12.5	6.85	2.38
Manggachapui (<i>Hopea</i> spp.)	20.3	7.58	12.6	4.75	2.19
Molave (<i>Vitex parviflora</i> Juss.)	18.9	5.15	12.1	5.00	2.27
Narig (<i>Vatica</i> spp.)	17.2	6.56	10.8	3.92	2.06
Sasalit [<i>Teijmanniodendron ahernianum</i> (Merr) Bkh.]	24.7	7.65	17.0	8.07	2.67
Yakal (<i>Shorea</i> spp.)	19.3	7.70	12.0	4.94	1.96
II. Moderately High Strength Group					
Antipolo (<i>Arthocarpus</i> spp.)	14.7	4.21	8.53	3.07	1.62
Binggas (<i>Terminalia</i> spp.)	14.9	5.17	8.98	2.57	1.77
Bokbok (<i>Xanthophyllum excelsum</i> (Blume) Miq.)	14.3	5.01	8.90	2.68	1.72
Dao (<i>Dracontomelon</i> spp.)	12.8	4.28	7.43	1.79	1.51
Gatasan [<i>Garcinia venulosa</i> (Blanco) Choisy]	16.4	5.39	10.6	2.77	1.86
Guijo (<i>Shorea</i> spp.)	17.1	6.67	10.4	3.35	1.89
Kamagong (<i>Diospyros</i> spp.)	16.6	5.67	9.21	3.46	1.95
Kamatog [<i>Erythrophloeum densiflorum</i> (Elm) Merr.]	15.0	5.95	8.79	3.11	1.85
Katmon (<i>Dillenia</i> spp.)	14.8	5.37	9.38	3.81	1.80
Kato (<i>Amoora</i> spp.)	14.5	6.33	8.34	2.73	1.54
Lomarau (<i>Swintonia foxworthyi</i> Elm.)	15.6	6.24	9.30	2.34	1.71
Mahogany, Big-leafed (<i>Swintonia macrophylla</i> King)	13.0	3.67	8.24	3.01	2.13
Makaasim (<i>Syzygium nitidum</i> Benth)	16.1	5.29	8.95	2.92	1.89
Malakauayan [<i>Decusocarpus philippinensis</i> (Foxw.) de Laub.]	14.9	5.24	8.79	1.83	1.69
Narra (<i>Pterocarpus indicus</i> Willd)	14.2	4.68	8.97	2.42	1.51
Pahunan (<i>Mangilera</i> spp.)	13.1	5.15	7.88	1.97	1.61
III. Medium Strength Group					
Apitong (<i>Dipterocarpus</i> spp.)	13.0	5.76	7.53	1.73	1.36
Bagtikan [<i>Parashorea malaanonan</i> (Blanco) Merr.]	13.1	5.10	7.79	1.84	1.43
Dangkalan (<i>Calophyllum</i> spp.)	12.8	5.03	7.24	1.96	1.56
Gisau (<i>Canarium</i> spp.)	11.2	4.20	6.43	1.56	1.49
Lanutan-bagyo [<i>Gonystylus macrophyllum</i> (miq.) Airy Shaw]	11.8	4.77	7.06	1.59	1.45
Lauan (<i>Shorea</i> spp.)	10.9	4.59	6.44	1.35	1.17
Malaanonang (<i>Shorea</i> spp.)	10.9	4.26	6.72	1.54	1.25
Malasaging (<i>Aglaiia</i> spp.)	13.3	4.68	7.49	2.30	1.46
Malugai (<i>Pometia</i> spp.)	12.1	4.96	7.35	2.42	1.63
Miau (<i>Dysoxylum</i> spp.)	12.3	5.12	6.96	2.19	1.62
Nato (<i>Palaquium</i> spp.)	12.7	4.38	7.22	1.84	1.56
Palosapis (<i>Anisoptera</i> spp.)	10.9	4.71	6.60	2.15	1.33
Pine (<i>Pinus</i> spp.)	11.6	5.24	6.53	1.48	1.23
Salakin (<i>Aphanamixis</i> spp.)	12.4	4.47	6.96	2.32	1.48
Vidal lanutan [<i>Hibiscus campylosiphon</i> Turcz. var. <i>glabrecens</i> (Har. Ex. Perk.)]	15.4	4.59	6.73	2.09	1.88
IV. Moderately Low Strength Group					
Almaciga [<i>Agathis dammara</i> (Lamb.) Rilh.]	9.26	4.30	4.94	1.13	1.16
Bayok (<i>Pterospermum</i> spp.)	9.94	3.74	5.78	1.03	0.95
Lingo-lingo (<i>Vitex turczaninowii</i> Merr.)	10.4	3.25	5.39	1.58	1.31
Mangasinoro (<i>Shorea</i> spp.)	10.0	4.22	5.87	1.55	1.14
Raintree [<i>Samanea saman</i> (Jacq.) Merr.]	9.37	2.16	5.70	2.61	1.63
Yemane (<i>Gmelina arborea</i> R. Br.)	9.90	3.22	6.20	2.68	1.55

¹ See Table 615.2-2 for Reference Values for Other Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods

Table 615.2-1 (Cont'd) Reference Values for Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods¹

Species (Common and Botanical Names)	50% Stress Grade				
	Bending and Tension Parallel to Grain	Modulus of Elasticity in Bending	Compression Parallel to Grain	Compression Perpendicular to Grain	Shear Parallel to Grain
	F_b, F_t	E	F_c	$F_{c\perp}$	F_v
	MPa	$\times 10^3$ MPa	MPa	MPa	MPa
I. High Strength Group					
Agoho (<i>Casuarina equisetifolia</i> Forst)	16.4	5.14	9.06	3.69	1.84
Liusin [<i>Parinari corymbosa</i> (Blume) Miq.]	15.6	5.85	9376	2.69	1.65
Malabayabas (<i>Tristania</i> spp.)	17.9	5.19	9390	5.44	1.89
Manggachapui (<i>Hopea</i> spp.)	16.1	6.02	10.0	3.77	1.74
Molave (<i>Vitex parviflora</i> Juss.)	15.0	4.09	9.60	3.96	1.80
Narig (<i>Vatica</i> spp.)	13.6	5.20	8.59	3.11	1.63
Sasalit [<i>Tejmanniodendron ahernianum</i> (Merr) Bkh.]	19.6	6.08	13.5	6.40	2.12
Yakal (<i>Shorea</i> spp.)	15.3	3.11	9.55	3.92	1.55
II. Moderately High Strength Group					
Antipolo (<i>Artocarpus</i> spp.)	11.6	3.34	6.77	2.44	1.29
Binggas (<i>Terminalia</i> spp.)	11.8	4.11	7.13	2.04	1.40
Bokbok (<i>Xanthophyllum excelsum</i> (Blume) Miq.)	11.3	3.97	7.06	2.13	1.36
Dao (<i>Dracontomelon</i> spp.)	10.1	3.39	5.90	1.42	1.20
Gatasan [<i>Garcinia venulosa</i> (Blanco) Choisy]	13.0	4.27	8.42	2.20	1.47
Guijo (<i>Shorea</i> spp.)	13.6	5.30	8.22	2.66	1.50
Kamagong (<i>Diospyros</i> spp.)	13.1	4.50	7.31	2.74	1.54
Kamatog [<i>Erythrophloeum densiflorum</i> (Elm) Merr.]	11.9	4.72	6.98	2.47	1.47
Katmon (<i>Dillenia</i> spp.)	11.7	4.26	7.44	3.03	1.43
Kato (<i>Amoora</i> spp.)	11.5	5.02	6.62	2.17	1.23
Lomarau (<i>Swintonia foixworthyi</i> Elm.)	12.4	4.95	7.38	2.86	1.36
Mahogany, Big-leafed (<i>Swintonia macrophylla</i> King)	10.3	2.91	6.54	2.39	1.69
Makaasim (<i>Syzygium nitidum</i> Benth)	12.8	4.20	7.10	2.31	1.50
Malakauayan [<i>Decusocarpus philippinensis</i> (Foxw.) de Laub.]	11.8	4.16	6.98	1.45	1.34
Narra (<i>Pterocarpus indicus</i> Willd)	11.2	3.71	7.12	1.92	1.20
Puhutan (<i>Mangifera</i> spp.)	10.4	4.08	6.25	1.56	1.28
III. Medium Strength Group					
Apitong (<i>Dipterocarpus</i> spp.)	10.3	4.57	5.97	1.37	1.08
Bagtikan [<i>Parashorea malaanonan</i> (Blanco) Merr.]	10.4	4.05	6.18	1.46	1.14
Dangkalan (<i>Calophyllum</i> spp.)	10.2	3.99	5.75	1.55	1.24
Gisau (<i>Canarium</i> spp.)	8.93	3.33	5.10	1.24	1.18
Lanutan-bagyo [<i>Gonystylus macrophyllum</i> (miq.) Airy Shaw]	9.39	3.79	5.60	1.26	1.15
Lauan (<i>Shorea</i> spp.)	8.68	3.64	5.11	1.07	0.93
Malaanonang (<i>Shorea</i> spp.)	8.63	3.38	5.34	1.23	0.99
Malasaging (<i>Aglaia</i> spp.)	10.5	3.71	5.95	1.83	1.16
Malugai (<i>Pometia</i> spp.)	9.62	3.94	5.83	1.92	1.30
Miau (<i>Dysoxylum</i> spp.)	9.80	4.06	5.52	1.74	1.29
Nato (<i>Palaquium</i> spp.)	10.1	3.48	5.73	1.46	1.24
Palosapis (<i>Anisoptera</i> spp.)	8.65	3.73	5.24	1.70	1.05
Pine (<i>Pinus</i> spp.)	9.19	4.16	5.18	1.18	0.98
Salakin (<i>Aphanamixis</i> spp.)	9.83	3.54	5.52	1.84	1.18
Vidal lanutan [<i>Hibiscus campylosiphon</i> Turcz. var. <i>glabrecens</i> (Har. Ex. Perk.)]	12.2	3.64	5.34	1.66	1.50
IV. Moderately Low Strength Group					
Almaciga [<i>Agathis dammara</i> (Lamb.) Rilh.]	7.35	3.42	3.92	0.90	0.92
Bayok (<i>Pterospermum</i> spp.)	7.89	2.97	4.58	0.81	0.75
Lingo-lingo (<i>Vitex turczaninowii</i> Merr.)	8.27	2.58	4.28	1.25	1.04
Mangasinoro (<i>Shorea</i> spp.)	7.98	3.35	4.66	1.23	0.90
Raintree [<i>Samanea saman</i> (Jacq.) Merr.]	7.43	1.72	4.52	2.07	1.30
Yemane (<i>Gmelina arborea</i> R. Br.)	7.86	2.55	4.92	2.13	1.23

¹ See Table 615.2-2 for Reference Values for Other Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods

Table 615.2-2 Reference Values for Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods^{1,2}

Additional Species (Common and Botanical Names)	80% STRESS GRADE					50% STRESS GRADE				
	F_b, F_t	E	F_c	$F_{c\perp}$	F_v	F_b, F_t	E	F_c	$F_{c\perp}$	F_v
	MPa	GPa	MPa	MPa	MPa	MPa	GPa	MPa	MPa	MPa
I. High Strength Group										
A. Commercial Species										
Alupag amo [Litchi chinensis Sonn. ssp. philippinensis (Radlk) Leenh.]	28.13	7.98	9.74	6.39	3.49	17.58	4.99	6.09	4.00	2.18
Ata-ata (Diospyros mindanaensis Merr.)	27.00	8.28	-	-	-	16.87	5.18	-	-	-
Bakauan (Rhizophora apiculata Blume)	31.45	9.60	8.90	6.30	3.05	19.65	6.00	5.56	3.93	1.90
Katilma (Diospyros nitida Merr)	26.17	8.31	8.03	3.54	2.66	16.35	5.19	5.02	2.21	1.66
Kubi (Artocarpus nitidus Trecc. spp. nitidus)	31.21	7.92	11.04	5.47	2.85	19.51	4.95	6.90	3.42	1.78
Narig (Vatica mangachapoi Blanco ssp. mangachapoi)	24.47	8.15	9.13	4.19	2.44	15.29	5.09	5.70	2.62	1.53
Narig, Thick leafed (Vatica pechyphylla Merr)	27.04	8.34	10.29	6.42	2.89	16.90	5.21	6.43	4.01	1.81
Tiga [Tristeniopsis micrantha (Merr) Wils. & Waterh.]	31.92	9.00	10.18	9.71	3.41	19.95	5.63	6.36	6.07	2.13
Tindalo [Afzelia rhomboidea (Blanco) Vid.]	30.17	8.88	11.39	7.56	3.41	18.86	5.55	7.12	4.73	2.13
Yakal (Shorea astylosa Foxw.)	25.05	9.92	10.14	6.72	2.33	15.66	6.20	6.34	4.20	1.46
Yakal-yamban (Shorea falciferoides ssp. falciferoides)	29.17	9.50	9.81	6.31	2.43	18.23	5.94	6.13	3.94	1.52
Yakal-malibato (Shorea malibato Foxw)	38.22	8.70	10.14	6.15	2.64	23.89	5.44	6.34	3.84	1.65
Yakal-saplungan [Hopea plagata (Blanco) Vid.]	41.79	11.18	12.76	9.00	2.91	26.12	6.99	7.97	5.62	1.82
Diospyros sp.	24.67	7.98	8.48	5.71	2.64	15.42	4.99	5.30	3.57	1.65
B. Lesser-Known Species										
Antsoan (Cassia javanica L. ssp. javanica)	27.47	7.62	9.44	5.08	3.00	17.17	4.76	5.90	3.17	1.87
Arangen [Ganophyllum obliquum (Blanco) Merr.]	26.85	7.86	9.82	6.88	3.00	16.78	4.91	6.14	4.30	1.87
Bansilai (Ochna foxworthyi Elm)	27.99	9.24	10.72	8.00	2.88	17.49	5.78	6.70	5.00	1.80
Satinwood (Chloroxylon swietenia DC.)	32.92	8.40	11.39	5.47	3.09	20.57	5.25	7.12	3.42	1.93

¹ Additional Species to Table 615.2-1² Linear interpolation between values for 80% Stress Grade and 50% Stress Grade may be used to determine values for the 63% Stress Grade

Table 615.2-2 (Cont'd) Reference Values for Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods^{1,2}

Additional Species (Common and Botanical Names)	80% STRESS GRADE					50% STRESS GRADE				
	F_b, F_t	E	F_c	$F_{c\perp}$	F_v	F_b, F_t	E	F_c	$F_{c\perp}$	F_v
	MPa	GPa	MPa	MPa	MPa	MPa	GPa	MPa	MPa	MPa
II. Moderately High Strength Group										
A. Commercial Species										
Akle [<i>Albizia acle</i> (Blanco) Merr.]	21.08	7.20	7.36	4.78	2.78	13.17	4.50	4.60	2.99	1.74
Amugis [<i>Koordersiodendron pinnatum</i> (Blanco) Merr.]	19.48	6.71	6.92	3.90	2.42	12.17	4.20	4.33	2.43	1.51
Anang (<i>Diospyros pyrrhcorpa</i> Miq.)	22.00	7.02	7.62	3.05	2.49	13.75	4.39	4.76	1.91	1.56
Anang-gulod [<i>Diospyros myrmecocalyx</i> (Hiern) Bakh]	20.20	6.15	7.41	3.61	2.57	12.62	3.85	4.63	2.26	1.61
Batino (<i>Alstonia macrophylla</i> G. Don)	-	-	-	-	-	-	-	-	-	-
Bingas [<i>Terminalia citrina</i> (Gaertn.) Roxb. ex. Flem.]	18.01	7.20	8.07	4.34	2.72	11.26	4.50	5.05	2.71	1.70
Bolon [<i>Platymitra arborea</i> (Blanco) Kesler]	25.34	7.62	8.38	3.63	2.29	15.84	4.76	5.24	2.27	1.43
Bolong-eta (<i>Diospyros pilosanthera</i>)	20.46	8.28	5.50	-	-	12.79	5.18	3.44	-	-
Blanco var <i>philosanthera</i>]	19.35	7.38	6.38	5.37	2.68	12.09	4.61	3.99	3.36	1.68
Dungon (<i>Heritiera sylvatica</i> Vidal)	23.73	7.20	9.52	6.15	2.72	14.83	4.50	5.95	3.84	1.70
Dysoxylum sp.	17.10	6.18	6.88	2.38	2.00	10.69	3.86	4.30	1.49	1.25
Ipil [<i>Intsia bijuga</i> (Colebr.) O. Ktze]	27.09	7.44	-	5.37	-	16.93	4.65	-	3.36	-
Kamagong (<i>Diospyros discolor</i> Willd.)	25.04	7.74	8.54	6.49	3.00	15.65	4.84	5.34	4.06	1.87
Kamagong ponce (<i>Diospyros poncei</i> Merr.)	22.00	6.30	6.86	4.82	2.19	13.75	3.94	4.29	3.01	1.37
Katmon-bayani (<i>Dillenia megalantha</i> Merr.)	21.17	7.56	6.46	4.51	2.13	13.23	4.73	4.04	2.82	1.33
Katong-lakihan (<i>Dysoxylum crytobotryum</i> Miq.)	19.56	8.22	6.69	3.42	1.90	12.22	5.14	4.18	2.14	1.19
Lithocarpus sp.	18.52	8.04	6.30	3.98	1.49	11.57	5.03	3.94	2.49	0.93
Ludek [<i>Ludekia bernardoi</i> (Merr.) Ridsd.]	24.30	7.44	6.78	4.18	2.56	15.18	4.65	4.24	2.61	1.60
Malakatmon [<i>Dillenia luzoniensis</i> (Vidal) Martelli ex Dur. et Jacks.]	23.60	7.40	7.37	5.40	2.46	14.75	4.63	4.61	3.38	1.54
Malapanau (<i>Dipterocarpus kerrii</i> King)	17.26	6.86	6.12	2.09	1.70	10.79	4.29	3.83	1.31	1.07
Malugai (<i>Pometia pinnata</i> Forst & Forst.)	17.10	6.36	5.99	2.80	2.15	10.69	3.98	3.75	1.75	1.34
Malugai-liitan (<i>Pometia pinnata</i> forma <i>repanda</i> Jacobs)	17.06	6.05	6.38	3.93	2.36	10.67	3.78	3.99	2.45	1.47
Palak-palak (<i>Palaquium lanceolatum</i> Blanco)	21.43	7.85	7.29	2.64	1.88	13.39	4.90	4.56	1.65	1.18
Panau, leaf-tailed (<i>Dipterocarpus caudatus</i> Foxw.)	19.23	7.32	5.66	2.61	1.62	12.02	4.58	3.54	1.63	1.02
Pianga [<i>Ganua obovatifolia</i> (Merr.) Assem]	18.27	7.03	5.70	2.66	1.94	11.42	4.39	3.56	1.66	1.21
Sakat (<i>Terminalia nitens</i> Presl.)	23.63	5.66	-	2.77	-	14.77	3.54	-	1.73	-
Ulaian [<i>Lithocarpus llanosii</i> (A.DC.) Rehd.]	17.67	5.55	6.93	3.38	2.21	11.04	3.47	4.33	2.11	1.38
Talisai-gubat (<i>Terminalia foetidissima</i> Griff.)	21.79	7.32	6.88	2.83	2.06	13.62	4.58	4.30	1.77	1.28
Toog [<i>Petersianthus quadrialatus</i> (Merr.) Merr.]	19.70	6.53	6.77	2.74	2.03	12.31	4.08	4.23	1.71	1.27
Yakal kaliot (<i>Hopea malibato</i> Foxw.)	23.96	8.25	7.62	3.45	2.55	14.98	5.16	4.76	2.16	1.59
B. Lesser-Known Species										
Balakat [<i>Ziziphus talanai</i> (Blanco) Merr.]	16.89	4.87	8.03	3.02	2.17	10.55	3.05	5.02	1.88	1.36
Balikkikan (<i>Drypetes longifolia</i> (Blume) Pax & K Hoffm.)	24.82	7.74	7.62	3.78	2.08	15.51	4.84	4.76	2.36	1.30
Kalamansanai Group (<i>Neonauclea</i> sp.)	14.94	5.50	-	3.56	2.14	9.34	3.44	-	2.23	1.34
Kalingag (<i>Cinnamomum mercadoi</i> Vid.)	21.03	5.83	7.12	2.78	2.22	13.14	3.64	4.45	1.74	1.39
Kapulasan (<i>Nephelium mutabile</i> Blume)	16.36	6.47	-	3.68	2.26	10.22	4.04	-	2.30	1.41
Langil [<i>Albizia lebbek</i> (L.) Benth.]	22.69	6.66	8.35	5.12	2.78	14.18	4.16	5.22	3.20	1.74
Patangis [<i>Magnolia candollei</i> (Blume) Keng var. <i>candollei</i>]	21.03	7.08	6.34	3.56	2.61	13.14	4.43	3.96	2.23	1.63
Siar [<i>Peltophorum pterocarpum</i> (DC.) K. Heyne]	21.45	5.94	7.17	7.37	2.30	13.41	3.71	4.48	4.61	1.44
Uas [<i>Harpulia arborea</i> (Blanco) Radik]	17.05	5.39	5.66	3.02	2.13	10.66	3.37	3.54	1.89	1.33
C. Plantation Species										
Acacia crassicarpa A. Cunn. ex Benth	20.08	6.78	5.31	2.64	2.04	12.55	4.24	3.32	1.65	1.27
Acacia cincinnata	15.25	5.77	6.46	2.97	2.28	9.53	3.61	4.04	1.86	1.42
Banaba [<i>Lagerstroemia speciosa</i> (L.) Pers.]	16.72	5.36	5.92	3.81	2.27	10.45	3.35	3.70	2.38	1.42
Ipil-ipil, Giant [<i>Leucaena leucocephala</i> (Lam.) de wit]	15.54	5.43	5.50	3.20	2.50	9.71	3.40	3.44	2.00	1.56

¹ Additional Species to Table 615.2-1² Linear interpolation between values for 80% Stress Grade and 50% Stress Grade may be used to determine values for the 63% Stress Grade

Table 615.2-2 (Cont'd) Reference Values for Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods^{1,2}

Additional Species (Common and Botanical Names)	80% STRESS GRADE					50% STRESS GRADE				
	F_b, F_t	E	F_c	$F_{c\perp}$	F_v	F_b, F_t	E	F_c	$F_{c\perp}$	F_v
	MPa	GPa	MPa	MPa	MPa	MPa	GPa	MPa	MPa	MPa
III. Medium Strength Group										
A. Commercial Species										
Apitong (<i>Dipterocarpus grandiflorus</i> Blanco)	17.67	8.01	6.26	2.42	2.00	11.05	5.01	3.91	1.51	1.25
Apitong, Basilan (<i>Dipterocarpus eurychus</i> Miq.)	18.26	7.62	5.87	1.93	1.66	11.41	4.76	3.67	1.21	1.04
Apitong Broad-winged (<i>Dipterocarpus kunstleri</i> King)	16.57	6.76	5.51	1.90	1.59	10.36	4.23	3.44	1.19	1.00
Bitao (<i>Calophyllum inophyllum</i> L.)	14.16	6.30	4.80	1.59	1.38	8.85	3.94	3.00	0.99	0.86
Dagang (<i>Anisoptera aurea</i> Foxw.)	14.98	5.84	5.24	2.58	1.63	9.36	3.65	3.28	1.61	1.02
Hagakhak (<i>Dipterocarpus validus</i> Blume)	16.98	6.23	5.59	1.59	1.54	10.61	3.90	3.50	0.99	0.96
Kalumpit (<i>Terminalia microcarpa</i> Decne.)	18.52	4.60	6.11	3.32	2.47	11.57	2.87	3.82	2.08	1.54
Katmon (<i>Dillenia philippinensis</i> Rolfe)	18.38	5.16	6.40	3.79	2.28	11.48	3.22	4.00	2.37	1.42
Kuling-babui (<i>Dysoxylum excelsum</i> Blume)	14.49	5.72	5.73	1.61	1.76	9.06	3.57	3.58	1.01	1.10
Kuling-manuk [<i>Aglaia luzoniensis</i> (Vid.) Merr & Rolfe]	18.42	5.25	7.58	3.08	2.06	11.51	3.28	4.74	1.92	1.29
Lamio [<i>Dracontomelon edule</i> (Blanco) Skeels]	15.68	4.36	4.77	1.83	1.58	9.80	2.72	2.98	1.14	0.99
Lanipau (<i>Terminalia copelandii</i> Elm.)	15.63	5.94	6.11	1.84	1.58	9.77	3.71	3.82	1.15	0.99
Lokinai [<i>Dacrydium beccarii</i> Parl.]	15.44	3.35	3.65	2.09	1.70	9.65	2.10	2.28	1.31	1.06
Lamog (<i>Planchonia spectabilis</i> Merr.)	19.13	5.08	5.63	4.65	2.09	11.96	3.17	3.52	2.91	1.31
Magabuyo (<i>Celtis luzonica</i> Warb.)	11.51	4.01	5.06	2.12	1.78	7.19	2.50	3.16	1.32	1.11
Nato Villamil [<i>Pouteria villamilli</i> (Merr) Baehni]	16.34	5.06	5.57	2.59	2.06	10.21	3.17	3.48	1.62	1.29
Philippine maple (<i>Acer laurinum</i> Hassk. apud Hoveen & de Vriese)	15.77	6.30	7.46	1.86	1.88	9.86	3.94	4.66	1.17	1.17
Piling-liitan [<i>Canarium luzonicum</i> (Blume) A. Gray]	15.89	4.66	5.41	2.09	2.08	9.93	2.91	3.38	1.31	1.30
Tangile [<i>Shorea polysperma</i> (Blanco) Merr.]	15.83	6.23	5.50	1.79	1.54	9.89	3.89	3.44	1.12	0.96
B. Lesser-Known Species										
Anang (<i>Diospyros pyrrhocarpa</i> Miq.)	15.88	5.16	-	2.38	1.86	9.92	3.23	-	1.49	1.16
Amunat (<i>Oraphea cumingiana</i> Vid)	16.86	6.42	6.34	2.14	1.75	10.54	4.01	3.96	1.34	1.09
Apanit [<i>Mastixia pentandra</i> Blume ssp. <i>philippinensis</i> (Wang.) Matt.]	17.10	6.48	5.12	1.69	1.65	10.69	4.05	3.20	1.06	1.03
Balukanag [<i>Chisocheton cumingianus</i> (C.DC.) Harms]	16.24	6.60	5.57	2.43	1.60	10.15	4.13	3.48	1.52	1.00
Banai-banai [<i>Radermachera pinnata</i> (Blanco) Seem]	20.60	5.15	7.01	3.07	2.02	12.88	3.22	4.38	1.92	1.26
Bitanghol (<i>Calophyllum blancoi</i> Pl. & Tr.)	10.31	4.67	-	1.63	1.51	6.44	2.92	-	1.02	0.94
Dalung (<i>Phyllocladus hypophyllum</i> Hook f.)	20.51	5.70	6.88	2.55	2.04	12.82	3.56	4.30	1.59	1.27
Gapas-gapas [<i>Camptostemon philippinense</i> (Vid.) Becc.]	18.04	5.36	7.26	1.85	1.44	11.28	3.35	4.54	1.16	0.90
Itangan [<i>Weinmannia luzoniensis</i> Vid.]	13.92	4.85	4.74	1.67	1.62	8.70	3.03	2.96	1.05	1.01
Java sala [<i>Sloanea javanica</i> (Miq.) Koord & Val.]	17.85	5.36	6.30	2.24	2.00	11.16	3.35	3.94	1.40	1.25
Kangko [<i>Aphanamixis polystachya</i> Wall.) R.N. Parker]	16.08	5.71	5.73	2.83	1.89	10.05	3.57	3.58	1.77	1.18
Malakmalak [<i>Palaquium philippense</i> (Perr.) C.B. Rosb.]	13.37	4.70	5.71	2.05	1.80	8.36	2.94	3.57	1.28	1.13
Nato [<i>Palaquium luzoniense</i> (F. Vill.) Vid]	16.72	5.45	5.57	2.14	1.86	10.45	3.40	3.48	1.34	1.16
Pagsahingin-bulog (<i>Canarium asperum</i> Benth)	16.58	6.66	5.63	1.64	1.86	10.36	4.16	3.52	1.02	1.16
Panang (<i>Palaquium</i> sp.)	14.44	5.48	-	2.59	2.01	9.03	3.42	-	1.62	1.25

¹ Additional Species to Table 615.2-1² Linear interpolation between values for 80% Stress Grade and 50% Stress Grade may be used to determine values for the 63% Stress Grade

Table 615.2-2 (Cont'd) Reference Values for Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods^{1,2}

Additional Species (Common and Botanical Names)	80% STRESS GRADE					50% STRESS GRADE				
	F_b, F_t	E	F_c	$F_{c\perp}$	F_v	F_b, F_t	E	F_c	$F_{c\perp}$	F_v
	MPa	GPa	MPa	MPa	MPa	MPa	GPa	MPa	MPa	MPa
III. Medium Strength Group (cont'd)										
B. Lesser-Known Species (cont'd)										
Philippine chestnut [<i>Castanopsis philipinensis</i> (Blanco) Vid]	14.02	4.98	4.90	2.34	1.75	8.76	3.11	3.06	1.46	1.09
Sagimsim [<i>Syzygium brevistylum</i> (C.B. Rob.) Merr.]	16.87	4.03	-	2.21	2.03	10.54	2.52	-	1.38	1.27
Santiki [<i>Cleidion spiciflorum</i> (Bum. F.) Merr.]	15.72	4.25	4.48	3.18	-	9.83	2.66	2.80	1.99	-
<i>Syzygium</i> sp.	12.05	4.21	-	1.89	1.78	7.53	2.63	-	1.18	1.12
Tan-ag (<i>Kleinhovia hospita</i> L.)	15.34	4.48	5.95	2.47	1.91	9.59	2.80	3.72	1.55*	1.20
Ulaian [<i>Lithocarpus celebicus</i> (Miq.) Rehd.]	18.81	5.09	-	4.51	2.63	11.76	3.18	-	2.82	1.64
Usuang-saha (<i>Endiandra laxiflora</i> Merr.)	15.49	5.39	4.99	2.81	1.80	9.68	3.37	3.12	1.76	1.12
<i>Ternstroemia</i> sp.	19.51	5.32	6.94	2.65	2.12	12.20	3.33	4.34	1.66	1.33
C. Plantation Species										
Acacia mangium Willd.	15.51	5.60	7.36	2.49	1.95	9.70	3.50	4.60	1.56	1.22
Nangka (<i>Artocarpus heterophyllus</i> Lamk.)	20.55	5.26	7.04	2.04	1.92	12.85	3.29	4.40	1.27	1.20
River red gum (<i>Eucalyptus camaldulensis</i> Dehnh.)	16.22	6.14	-	3.49	1.51	10.14	3.84	-	2.18	0.94
Teak (<i>Tectona grandis</i> Lf.)	18.94	4.99	4.93	2.81	2.01	11.84	3.12	3.08	1.76	1.26

¹ Additional Species to Table 615.2-1² Linear interpolation between values for 80% Stress Grade and 50% Stress Grade may be used to determine values for the 63% Stress Grade

Table 615.2-2 (Cont'd) Reference Values for Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods^{1,2}

Additional Species (Common and Botanical Names)	80% STRESS GRADE					50% STRESS GRADE				
	F_b, F_t	E	F_c	$F_{c\perp}$	F_v	F_b, F_t	E	F_c	$F_{c\perp}$	F_v
	MPa	GPa	MPa	MPa	MPa	MPa	GPa	MPa	MPa	MPa
IV. Moderately Low Strength Group										
A. Commercial Species										
Almon (<i>Shorea almon</i> Foxw.)	15.32	6.14	5.48	1.63	1.41	9.57	3.83	3.43	1.02	0.88
Anubing (<i>Artocarpus ovatus</i> Blanco)	19.46	4.04	5.34	4.19	1.87	12.17	2.53	3.34	2.62	1.17
Batikuling (<i>Litsea leytensis</i> Merr.)	13.32	4.86	6.08	1.41	1.36	8.33	3.04	3.80	0.88	0.85
Dulit [<i>Canarium hirsutum</i>]	12.41	3.52	4.64	1.21	1.38	7.76	2.20	2.90	0.75	0.86
Willd. Forma multipinnatum (Llanos) H J. Lam.]	-	-	-	-	-	-	-	-	-	-
Igem [<i>Dacrydium imbricatum</i> (Bl.) var. patulus de Laub.]	12.17	4.95	4.26	1.24	1.45	7.61	3.09	2.66	0.77	0.91
Ilo-ilo [<i>Aglaia argentea</i> Blume]	16.86	5.50	4.83	1.50	1.42	10.54	3.44	3.02	0.94	0.89
Kalunti [<i>Shorea hopeifolia</i> (Heim) Sym]	14.11	4.97	5.50	1.61	1.44	8.82	3.11	3.44	1.00	0.90
Loktob (<i>Duabanga moluccana</i> Blume)	8.64	1.79	4.51	1.30	1.11	5.40	1.12	2.82	0.81	0.69
Manggasinoro [<i>Shorea assamica</i>]	14.11	5.65	5.13	1.54	1.40	8.82	3.53	3.21	0.96	0.87
Dyer, ssp. philippinensis (Brandis) Sym]	-	-	-	-	-	-	-	-	-	-
Manggasinorong-lakihan (<i>Shorea virescens</i> Parijs)	14.44	5.59	5.57	1.88	1.35	9.03	3.50	3.48	1.18	0.84
Mayapis [<i>Shorea palosapis</i> (Blanco) Merr.]	14.60	5.81	5.20	1.53	1.34	9.12	3.63	3.25	0.96	0.84
Paguringon (<i>Cratoxylum sumatranum</i> (Jack) Blume ssp. sumatranum Robs.].	15.77	4.76	6.05	2.32	2.06	9.86	2.97	3.78	1.45	1.29
Tuai (<i>Bischofia javanica</i> Blume)	14.75	4.14	4.66	4.10	1.80	9.22	2.59	2.91	2.56	1.13
Shorea sp.	13.92	4.84	4.26	1.04	1.15	8.70	3.03	2.66	0.65	0.72
B. Lesser-Known Species										
Anongo (<i>Turpinia ovalifolia</i> Elm.)	11.70	3.68	3.78	1.44	1.30	7.31	2.30	2.36	0.90	0.81
Balakot-gubat [<i>Sapium luzonicum</i> (Vid.) Merr.]	-	-	-	1.38	1.55	-	-	-	0.86	0.97
Balanti [<i>Homalanthus populneus</i> (Geisel.) Pax var. populneus]	11.32	3.83	-	1.65	1.31	7.07	2.39	-	1.03	0.82
Balete (<i>Ficus balete</i> Merr.)	13.84	5.20	-	2.30	1.72	8.65	3.25	-	1.44	1.08
Balobo (<i>Diplodiscus paniculatus</i> Turcz.)	16.59	4.71	-	3.12	2.02	10.37	2.94	-	1.95	1.26
Bayok (<i>Pterospermum diversifolium</i> Blume)	15.44	4.40	5.41	1.69	1.44	9.65	2.75	3.38	1.06	0.90
Bayok-bayokan (<i>Pterospermum niveum</i> Vid.)	13.56	4.69	5.62	1.45	1.29	8.48	2.93	3.51	0.91	0.81
Binunga [<i>Macaranga tanarius</i> (L.) Muell-Arg.]	9.79	3.39	-	1.50	1.37	6.12	2.12	-	0.94	0.85
Buta-buta (<i>Excoecaria agallocha</i> L.)	10.61	3.40	3.68	1.40	1.33	6.63	2.12	2.30	0.87	0.83
Duguan (<i>Myristica philippensis</i> Lam.)	8.39	4.38	-	1.92	1.38	5.25	2.74	-	1.20	0.86
Himbabao [<i>Broussonetia luzonica</i> (Blanco) Bur. var. luzonica]	12.83	4.04	4.45	2.24	2.00	8.02	2.53	2.78	1.40	1.25
Katong-matsin [<i>Chisocheton pentandrus</i> (Blanco) Merr.]	14.49	3.56	4.26	1.51	1.32	9.06	2.23	2.66	0.95	0.83
Tulo [<i>Alphitonia philippinensis</i> (Braid.) Gordon sp.]	14.11	3.81	4.61	1.63	1.54	8.82	2.38	2.88	1.02	0.97
C. Plantation Species										
Bagras (<i>Eucalyptus deglupta</i> Blume)	11.71	4.05	4.80	1.23	1.05	7.32	2.53	3.00	0.77	0.66
Durian (<i>Durio zibethinus</i> Merr.)	13.88	4.90	5.15	1.80	1.42	8.67	3.06	3.22	1.13	0.89
Para-rubber [<i>Hevea brasiliensis</i> (HBK) Muell-Arg.]	11.13	3.91	3.33	2.19	1.67	6.96	2.45	2.08	1.37	1.05
Santol [<i>Sandoricum koetjape</i> (Burm. f.) merr.]	11.84	2.91	3.62	1.44	1.31	7.40	1.82	2.26	0.90	0.82

¹ Additional Species to Table 615.2-1² Linear interpolation between values for 80% Stress Grade and 50% Stress Grade may be used to determine values for the 63% Stress Grade

Table 615.2-2 (Cont'd) Reference Values for Visually Stress-Graded Unseasoned Structural Timber of Philippine Woods^{1,2}

Additional Species (Common and Botanical Names)	80% STRESS GRADE					50% STRESS GRADE				
	F_b, F_t	E	F_c	$F_{c\perp}$	F_v	F_b, F_t	E	F_c	$F_{c\perp}$	F_v
	MPa	GPa	MPa	MPa	MPa	MPa	GPa	MPa	MPa	MPa
V. Low Strength Group										
A. Commercial Species										
Kalantas (<i>Toona calantas</i> Merr. & Rolfe)	9.66	3.70	3.04	0.86	0.88	6.04	2.31	1.90	0.54	0.55
Malakalumpang (<i>Sterculia</i>	7.72	3.74	3.58	1.08	0.91	4.82	2.34	2.24	0.68	0.57
Rarang [<i>Erythrina subumbrans</i> (Hassk) Merr.]	5.23	1.83	2.22	0.73	0.78	3.27	1.15	1.39	0.45	0.48
Tiaong (<i>Shorea ovata</i> Dyer ex Brandis)	11.56	4.97	4.02	0.91	1.07	7.22	3.11	2.51	0.57	0.67
Taluto [<i>Pterocymbium tinctorium</i> (Blanco) Merr.]	9.50	3.09	3.30	0.92	1.01	5.93	1.93	2.06	0.58	0.63
B. Lesser-Known Species										
Anabiong [<i>Trema orientalis</i> (L.)]	5.56	2.36	2.46	1.01	1.02	3.48	1.48	1.54	0.63	0.63
Bagalunga (<i>Melia azedarach</i> L.)	10.32	3.76	3.71	1.54	1.72	6.45	2.35	2.32	0.96	1.08
Banilad (<i>Sterculia camosa</i> Wall.)	6.07	2.15	-	0.83	0.81	3.79	1.34	-	0.52	0.50
Binuang (<i>Octomeles sumatrana</i> Miq.)	9.09	3.66	3.11	0.74	0.88	5.68	2.29	1.94	0.47	0.55
Dita [<i>Alstonia scholaris</i> (L.)]	4.64	1.91	-	0.55	0.80	2.90	1.20	-	0.34	0.50
R. Br. var. <i>scholarisis</i>]	10.14	3.42	3.84	1.34	0.86	6.33	2.14	2.40	0.84	0.54
Kaitana [<i>Zanthoxylum limonella</i> (Dennst.) Alston]	11.37	4.36	4.26	0.96	1.29	7.10	2.72	2.66	0.60	0.81
Tangisang-bayawak (<i>Ficus variegata</i> Blume var. <i>variegata</i>)	4.32	1.56	-	0.64	0.78	2.70	0.97	-	0.40	0.49
C. Plantation Species										
Alnus sp.	9.66	2.53	2.81	1.78	1.80	6.04	1.58	1.75	1.11	1.13
Balsa (<i>Ochroma pyramidale</i> (Cav.) Urb.)	8.76	2.77	3.65	1.30	-	5.48	1.73	2.28	0.81	-
Bayabas (<i>Psidium guajava</i> L.)	12.55	2.68	-	-	-	7.84	1.68	-	-	-
Ilang-ilang [<i>Cananga odorata</i> (Lam.) Hook f. & Thoms.]	-	-	-	-	-	-	-	-	-	-
Gubas (<i>Endospermum peltatum</i> Merr.)	9.66	2.96	3.74	2.36	1.02	6.04	1.85	2.34	1.48	0.64
Kaatoan bangkal (<i>Anthocephalus chinensis</i> (Lamk.) Rich. ex. Walp]	11.08	2.77	3.20	1.40	1.33	6.93	1.73	2.00	0.88	0.83
Kapok [<i>Ceiba pentandra</i> (L.) Gaertn.]	4.27	1.35	1.80	0.74	0.63	2.67	0.84	1.13	0.46	0.39
Lumbang [<i>Aleurites moluccana</i> (L.) Willd.]	6.39	2.47	1.63	0.71	0.88	4.00	1.55	1.02	0.44	0.55
Malapapaya [<i>Polyscias nodosa</i> (Blume) Seem]	10.92	4.04	5.25	0.97	1.17	6.82	2.53	3.28	0.61	0.73
Moluccan sau [<i>Paraserianthes falcata</i> (L.) Nielsen]	10.75	3.87	4.26	1.11	1.21	6.72	2.42	2.66	0.70	0.75
Spanish cedar (<i>Cedrela odorata</i> L.)	10.94	3.61	4.22	1.31	1.20	6.84	2.26	2.64	0.82	0.75
Tulip, African (<i>Spathodea campanulata</i> Beauv.)	6.06	1.63	2.33	0.85	0.98	3.79	1.02	1.46	0.53	0.61

¹ Additional Species to Table 615.2-1² Linear interpolation between values for 80% Stress Grade and 50% Stress Grade may be used to determine values for the 63% Stress Grade

Table 615.2-3 Limits of Defects by Grade in Joists and Planks for Seasoned Wood

Kind of Defects		Stress Grade					
		80%		63%		50%	
A. Natural Defects		Not permitted Not permitted 20 <					

¹ The size of knots on the narrow face within the middle third of length may be increased proportionately towards the ends of the piece of twice the size permitted on the narrow face but not to exceed that allowable along the center line of the wide face. The size of knots on the edge of wide face within the middle third of length may be increased proportionately towards the center of the wide face and towards the ends of the piece to the size permitted along the center line of the wide face. The sum of the sizes of all knots in any 150 mm of length of the piece shall not exceed twice the maximum permissible size of knots. Two knots of maximum size shall not be allowed in the same 150 mm of length on any face. Cluster knots and knots in group shall not be permitted.

Table 615.2-3 (cont'd) Limits of Defects by Grade in Joists and Planks for Seasoned Wood

Kind of Defects	Stress Grade		
	80%	63%	50%
B. Handling, Manufacture or Processing Defects			
1. Wane (maximum allowable size in mm)			
Nominal face dimension in mm			
50	3	12	12
75	3	12	12
100	6	15	15
125	6	15	15
150	10	20	20
200	12	22	25
250	15	25	30
300	18	28	38
350	20	38	45
400	25	45	50
450 and over	30	50	55
2. Torn grain (allowable depth in mm)	2	2	3
3. Skips, allowable size not to exceed:			
surface area (Width x length, mm)	width x 100	width x 100	width x 100
Depth (mm)	1	2	3
Quantity	1 skip per 5 m or shorter length	1 skip per 5 m or shorter length	1 skip per 5 m or shorter length

SECTION 616 DESIGN PROVISIONS AND EQUATIONS

616.1 General

616.1.1 Scope

This section establishes general design provisions that apply to all wood structural members and connections covered under this Specification. Each wood structural member or connection shall be of sufficient size and capacity to carry the applied loads without exceeding the adjusted design values specified herein.

616.1.2 Net Section Area

616.1.2.1 The net section area is obtained by deducting from the gross area the projected area of all material removed by boring, grooving, dapping, notching, or other means. The net section area shall be used in calculating the load carrying capacity of the member, except as specified in 616.6.3 for columns. The effects of any eccentricity of loads applied to the member at the critical net section shall be taken into account.

616.1.2.2 For parallel to grain loading with staggered bolts, drift bolts, drift pins, or lag screws, adjacent fasteners shall be considered as occurring at the same critical section if the parallel to grain spacing between fasteners in adjacent rows is less than four fastener diameters (see Figure 616.1-1).

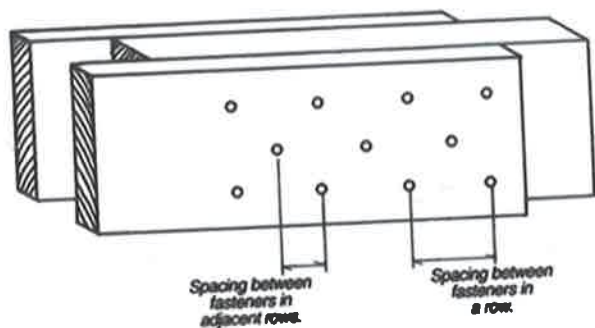


Figure 616.1-1 Spacing of Staggered Fasteners

616.1.2.3 The net section area at a split ring or shear plate connection shall be determined by deducting from the gross section area the projected areas of the bolt hole and

the split ring or shear plate groove within the member (see Figure 616.1-2). Where split ring or shear plate connectors are staggered, adjacent connectors shall be considered as occurring at the same critical section if the parallel to grain spacing between connectors in adjacent rows is less than or equal to one connector diameter (see Figure 616.1-1).

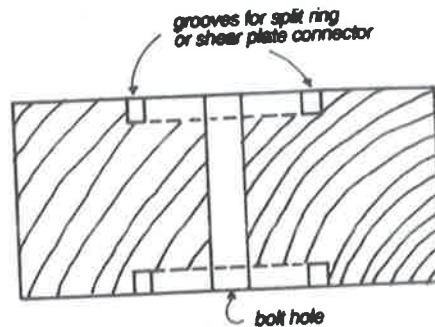


Figure 616.1-2 Net Cross Section at a Split Ring or Shear Plate Connection

616.1.3 Connections

Structural members and fasteners shall be arranged symmetrically at connections, unless the bending moment induced by an unsymmetrical arrangement (such as lapped joints) has been accounted for in the design. Connections shall be designed and fabricated to insure that each individual member carries its proportional stress.

616.1.4 Time Dependent Deformations

Where members of structural frames are composed of two or more layers or sections, the effect of time dependent deformations shall be accounted for in the design (see 616.5.2).

616.1.5 Composite Construction

Composite constructions, such as wood-concrete, wood-steel, and wood-wood composites, shall be designed in accordance with principles of engineering mechanics using the adjusted design values for structural members and connections specified herein.

616.2 Bending Members - General

616.2.1 Span of Bending Members

For simple, continuous and cantilevered bending members, the span shall be taken as the distance from face to face of support, plus one half the required bearing length at each end.

616.2.2 Lateral Distribution of Concentrated Load

Lateral distribution of concentrated loads from a critically loaded bending member to adjacent parallel bending members by flooring or other cross members shall be permitted to be calculated when determining design bending moment and vertical shear force.

616.2.3 Notches

616.2.3.1 Bending members shall not be notched except as permitted by 617.4.3 and 618.4.5. A gradual taper cut from the reduced depth of the member to the full depth of the member in lieu of a square-cornered notch reduces stress concentrations.

616.2.3.2 The stiffness of a bending member, as determined from its cross-section, is practically unaffected by a notch with the following dimensions:

$$\begin{array}{ll} \text{notch depth} & \leq (1/6)(\text{beam depth}) \\ \text{notch length} & \leq (1/3)(\text{beam depth}) \end{array}$$

616.2.3.3 See 616.4.3 for effect of notches on shear strength.

616.3 Bending Members – Flexure

616.3.1 Strength in Bending

The actual bending stress or moment shall not exceed the adjusted bending design value.

616.3.2 Flexural Design Equations

616.3.2.1 The actual bending stress induced by a bending moment, M , is calculated as follows:

$$f_b = \frac{Mc}{I} = \frac{M}{S} \quad (616-1)$$

For a rectangular bending member of breadth, b , and depth, d , this becomes:

$$f_b = \frac{M}{S} = \frac{6M}{bd^2} \quad (616-2)$$

616.3.2.2 For solid rectangular bending members with the neutral axis perpendicular to depth at center:

$$I = \frac{bd^3}{12} = \text{moment of inertia, mm}^4 \quad (616-3)$$

$$S = \frac{I}{c} = \frac{bd^2}{6} = \text{section modulus, mm}^3 \quad (616-4)$$

616.3.3 Beam Stability Factor, C_L

616.3.3.1 When the depth of a bending member does not exceed its breadth, $d \leq b$, no lateral support is required and $C_L = 1.0$.

616.3.3.2 When rectangular sawn lumber bending members are laterally supported in accordance with 617.4.1, $C_L = 1.0$.

616.3.3.3 When the compression edge of a bending member is supported throughout its length to prevent lateral displacement, and the ends at points of bearing have lateral support to prevent rotation, $C_L = 1.0$.

616.3.3.4 Where the depth of a bending member exceeds its breadth, $d > b$, lateral supports shall be provided at points of bearing to prevent rotation. When such lateral support is provided at points of bearing, but no additional lateral support is provided throughout the length of the bending member, the unsupported length, ℓ_u , is the distance between such points of end bearing, or the length of a cantilever. When a bending member is provided with lateral support to prevent rotation at intermediate points as well as at the ends, the unsupported length, ℓ_u , is the distance between such points of intermediate lateral support.

616.3.3.5 The effective span length, ℓ_e , for single span or cantilever bending members shall be determined in accordance with Table 616.3-1.

616.3.3.6 The slenderness ratio, R_B , for bending members shall be calculated as follows:

$$R_B = \sqrt{\frac{\ell_e d}{b^2}} \quad (616-5)$$

616.3.3.7 The slenderness ratio for bending members, R_B , shall not exceed 50.

616.3.3.8 When lateral support to compression side of beam may permit beam to buckle laterally, the beam stability factor, C_L , shall be calculated as follows:

$$C_L = \frac{1 + (F_{bE}/F_b^*)}{1.9} - \sqrt{\left[\frac{1 + (F_{bE}/F_b^*)}{1.9} \right]^2 - \frac{F_{bE}/F_b^*}{0.95}} \quad (616-6)$$

where:

F_b^* = Reference bending design value multiplied by all applicable adjustment factors except C_{fu} , C_V , and C_L , MPa

F_{bE} = Euler-based WSD critical buckling value for bending members

$$= \frac{1.20E_{min}'}{R_B^2}$$

616.3.3.9 Members subjected to flexure about both principal axes (biaxial bending) shall be designed in accordance with 616.9.2.

Table 616.3-1 Effective Length, ℓ_e , for Bending Members

Cantilever ¹	where $\ell_u/d < 7$	where $\ell_u/d \geq 7$
Uniformly distributed load	$\ell_e = 1.33\ell_u$	$\ell_e = 0.90\ell_u + 3d$
Concentrated load at unsupported end	$\ell_e = 1.87\ell_u$	$\ell_e = 1.44\ell_u + 3d$
Single Span Beam ^{1,2}	where $\ell_u/d < 7$	where $\ell_u/d \geq 7$
Uniformly distributed load	$\ell_e = 2.06\ell_u$	$\ell_e = 1.63\ell_u + 3d$
Concentrated load at center with no intermediate lateral support	$\ell_e = 1.80\ell_u$	$\ell_e = 1.37\ell_u + 3d$
Concentrated load at center with lateral support at center	$\ell_e = 1.11\ell_u$	
Two equal concentrated loads at 1/3 points with lateral support at 1/3 points	$\ell_e = 1.68\ell_u$	
Three equal concentrated loads at 1/4 points with lateral support at 1/4 points	$\ell_e = 1.54\ell_u$	
Four equal concentrated loads at 1/5 points with lateral support at 1/5 points	$\ell_e = 1.68\ell_u$	
Five equal concentrated loads at 1/6 points with lateral support at 1/6 points	$\ell_e = 1.73\ell_u$	
Six equal concentrated loads at 1/7 points with lateral support at 1/7 points	$\ell_e = 1.78\ell_u$	
Seven or more equal concentrated loads, evenly spaced, with lateral support at points of load application	$\ell_e = 1.84\ell_u$	
Equal end moments	$\ell_e = 1.84\ell_u$	

¹ For single span or cantilever bending members with loading conditions not specified in Table 616.3-1:

$$\ell_e = 2.06\ell_u \quad \text{where } \ell_u/d < 7$$

$$\ell_e = 1.63\ell_u + 3d \quad \text{where } 7 \leq \ell_u/d \leq 14.3$$

$$\ell_e = 1.84\ell_u \quad \text{where } \ell_u/d > 14.3$$

² Multiple span applications shall be based on table values or engineering analysis.

616.4 Bending Members – Shear

616.4.1 Strength in Shear Parallel to Grain (Horizontal Shear)

616.4.1.1 The actual shear stress parallel to grain or shear force at any cross section of the bending member shall not exceed the adjusted shear design value. A check of the strength of wood bending members in shear perpendicular to grain is not required.

616.4.1.2 The shear design procedures specified here in for calculating f_v at or near points of vertical support are limited to solid flexural members such as sawn lumber, structural glued laminated timber, structural composite lumber, or mechanically laminated timber beams. Shear design at supports for built-up components containing load-bearing connections at or near points of support, such as between the web and chord of a truss, shall be based on test or other techniques.

616.4.2 Shear Design Equations

The actual shear stress parallel to grain induced in a sawn lumber, structural glued laminated timber, structural composite lumber, or timber pole or pile bending member shall be calculated as follows:

$$f_v = \frac{VQ}{Ib} \quad (616-7)$$

For a rectangular bending member of breadth, b , and depth, d , this becomes:

$$f_v = \frac{3V}{2bd} \quad (616-8)$$

616.4.3 Shear design

616.4.3.1 When calculating the shear force, V , in bending members:

(a) For beams supported by full bearing on one surface and loads applied to the opposite surface, uniformly distributed loads within a distance from supports equal to the depth of the bending member, d , shall be permitted to be ignored. For beams supported by full bearing on one surface and loads applied to the opposite surface, concentrated loads within a distance, d , from supports shall be permitted to be multiplied by x/d where x is the distance from the beam support face to the load (see Figure 616.4-1).

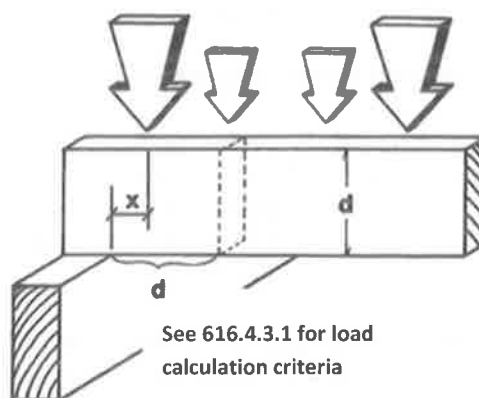


Figure 616.4-1 Shear at Supports

(b) The largest single moving load shall be placed at a distance from the support equal to the depth of the bending member, keeping other loads in their normal relation and neglecting any load within a distance from a support equal to the depth of the bending member. This condition shall be checked at each support.

(c) With two or more moving loads of about equal weight and in proximity, loads shall be placed in the position that produces the highest shear force, V , neglecting any load within a distance from a support equal to the depth of the bending member.

616.4.3.2 For notched bending members, shear force, V , shall be determined by principles of engineering mechanics (except those given in 616.4.3.1).

(a) For bending members with rectangular cross section and notched on the tension face (see 616.2.3), the adjusted design shear, V_r' , shall be calculated as follows:

$$V_r' = \left[\frac{2}{3} F_v' b d_n \right] \left[\frac{d_n}{d} \right]^2 \quad (616-9)$$

where:

- d = depth of unnotched bending member, mm
- d_n = depth of member remaining at a notch measured perpendicular to length of member, mm
- F_v' = adjusted shear design value parallel to grain, MPa

(b) For bending members with circular cross section and notched on the tension face (see 616.2.3), the adjusted design shear, V_r' , shall be calculated as follows:

$$V_r' = \left[\frac{2}{3} F_v' A_n \right] \left[\frac{d_n}{d} \right]^2 \quad (616-10)$$

where:

A_n = cross sectional area of notched member, mm²

(c) For bending members with other than rectangular or circular cross section and notched on the tension face (see 616.2.3), the adjusted design shear, V_r' , shall be based on conventional engineering analysis of stress concentrations at notches.

(d) A gradual change in cross section compared with a square notch decreases the actual shear stress parallel to grain nearly to that computed for an unnotched bending member with a depth of d_n .

(e) When a bending member is notched on the compression face at the end as shown in Figure 616.4-2, the adjusted design shear, V_r' , shall be calculated as follows:

$$V_r' = \frac{2}{3} F_v' b \left[d - \left(\frac{d - d_n}{d_n} \right) e \right] \quad (616-11)$$

where:

e = the distance the notch extends from the inner edge of the support and must be less than or equal to the depth remaining at the notch, $e \leq d_n$. If $e > d_n$, d_n shall be used to calculate f_v using Equation 616-8

d_n = depth of member remaining at a notch meeting the provisions of 616.2.3, measured perpendicular to length of member. If the end of the beam is beveled, as shown by the dashed line in Figure 616.4-2, d_n is measured from the inner edge of the support, mm

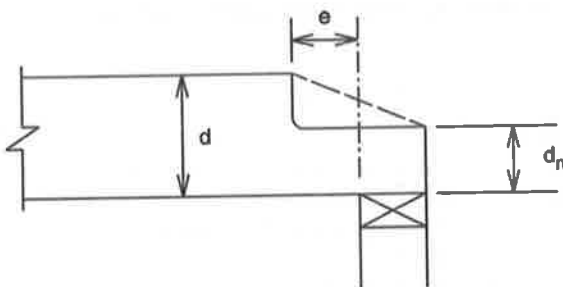


Figure 616.4-2 Bending Member End-Notched at Compression Face

616.4.3.3 When connections in bending members are fastened with split ring connectors, shear plate connectors, bolts, or lag screws (including beams

supported by such fasteners or other cases as shown in Figures 616.4-3 and 616.10-1) the shear force, V , shall be determined by principles of engineering mechanics (except those given in 616.4.3.1)

(a) Where the connection is less than five times the depth, $5d$, of the member from its end, the adjusted design shear, V_r' , shall be calculated as follows:

$$V_r' = \left[\frac{2}{3} F_v' b d_e \right] \left[\frac{d_e}{d} \right]^2 \quad (616-12)$$

where:

for split ring or shear plate connections:

d_e = depth of member, less the distance from the unloaded edge of the member to the nearest edge of the nearest split ring or shear plate connector (see Figure 616.4-3), mm.

for bolt or lag screw connections:

d_e = depth of member, less the distance from the unloaded edge of the member to the center of the nearest bolt or lag screw (see Figure 616.4-3), mm.

(b) Where the connection is at least five times the depth, $5d$, of the member from its end, the adjusted design shear, V_r' , shall be calculated as follows:

$$V_r' = \frac{2}{3} F_v' b d_e \quad (616-13)$$

(c) Where concealed hangers are used, the adjusted design shear, V_r' , shall be calculated based on the provisions in 616.4.3.2 for notched bending members.

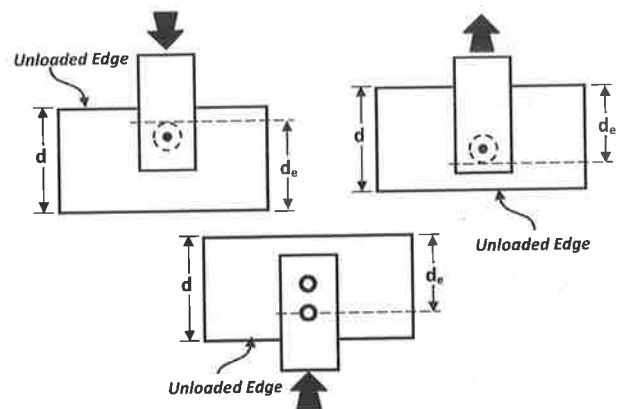


Figure 616.4-3 Effective depth, d_e , of Members at Connections

616.5 Bending Members - Deflection

616.5.1 Deflection Calculations

If deflection is a factor in design, it shall be calculated by standard methods of engineering mechanics considering bending deflections and, when applicable, shear deflections. Consideration for shear deflection is required when the reference modulus of elasticity has not been adjusted to include the effects of shear deflection.

616.5.2 Long-Term Loading

Where total deflection under long-term loading must be limited, increasing member size is one way to provide extra stiffness to allow for this time dependent deformation. Total deflection, Δ_T , shall be calculated as follows:

$$\Delta_T = K_{cr} \Delta_{LT} + \Delta_{ST} \quad (616-13)$$

where:

- K_{cr} = time dependent deformation (creep) factor.
- = 1.5 for seasoned lumber and structural glued laminated timber used in dry service conditions as defined in 617.1.4 and 618.1.4, respectively.
 - = 2.0 for structural glued laminated timber used in wet service conditions as defined in 618.1.4.
 - = 2.0 for unseasoned lumber or for seasoned lumber used in wet service conditions as defined in 617.1.4
- Δ_{LT} = immediate deflection due to the term component of the design load, mm.
- Δ_{ST} = deflection due to the short-term or normal component of the design load, mm.

616.6 Compression Members - General

616.6.1 Terminology

For purposes of this Specification, the term "column" refers to all types of compression members, including members forming part of trusses or other structural components.

616.6.2 Simple Solid Wood Columns

Simple columns consist of a single piece or of pieces properly glued together to form a single member (see Figure 616.6-1).

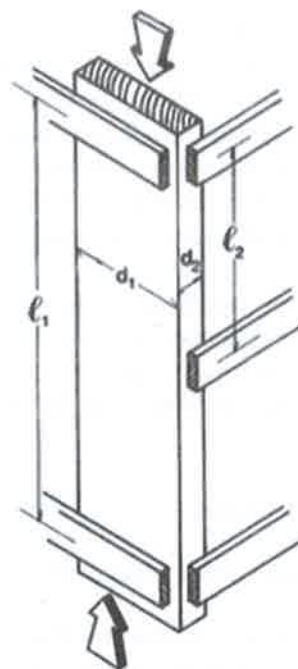


Figure 616.6-1 Simple Solid Columns

616.6.3 Strength in Compression Parallel to Grain

The actual compression stress or force parallel to grain shall not exceed the adjusted compression design value. Calculations of f_c shall be based on the net section area (see 616.1.2) where the reduced section occurs in the critical part of the column length that is most subject to potential buckling. Where the reduced section does not occur in the critical part of the column length that is most subject to potential buckling, calculations of f_c shall be based on gross section area. In addition, f_c based on net section area shall not exceed the reference compression design value parallel to grain multiplied by all applicable adjustment factors except the column stability factor, C_P .

616.6.4 Compression Members Bearing End to End

For end grain bearing of wood on wood, and on metal plates or strips see 616.10.

616.6.5 Eccentric Loading or Combined Stresses

For compression members subject to eccentric loading or combined flexure and axial loading, see 616.9.

616.6.6 Column Bracing

Column bracing shall be installed where necessary to resist wind or other lateral forces. In buildings, for forces acting in a direction parallel to the truss or beam, column bracing shall be permitted to be provided by knee braces or, in the case of trusses, by extending the column to the top chord of the truss where the bottom and top of chords are separated sufficiently to provide adequate bracing action. In a direction perpendicular to the truss or beam, bracing shall be permitted to be provided by wall construction, knee braces, or bracing between columns. Such bracing between columns should be installed preferably in the same bays as the bracing between trusses.

616.7 Solid Columns

616.7.1 Column Stability Factor, C_P

616.7.1.1 When a compression member is supported throughout its length to prevent lateral displacement in all directions, $C_P = 1.0$.

616.7.1.2 The effective column length, ℓ_e , for a solid column shall be determined in accordance with principles of engineering mechanics. One method for determining effective column length, when end-fixity conditions are known, is to multiply actual column length by the appropriate effective length factor or buckling length coefficients, $\ell_e = (K_e)(\ell)$. (see Figure 616.7-1 for values of K_e).

616.7.1.3 For solid columns with rectangular cross section, the slenderness ratio, ℓ_e/d , shall be taken as the larger of the ratios ℓ_{e1}/d_1 or ℓ_{e2}/d_2 (see Figure 616.6-1) where each ratio has been adjusted by the appropriate buckling length coefficient, K_e , from Figure 616.7-1.

Buckling modes						
Theoretical K_e value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design K_e when ideal conditions approximated	0.65	0.80	1.2	1.0	2.10	2.4
End condition code						
	Rotation fixed, translation fixed Rotation free, translation fixed Rotation fixed, translation free Rotation free, translation free					

Figure 616.7-1 Buckling Length Coefficients, K_e

616.7.1.4 The slenderness ratio for solid columns, ℓ_e/d , shall not exceed 50, except that during construction ℓ_e/d shall not exceed 75.

616.7.1.5 The column stability factor shall be calculated as follows:

$$C_P = \frac{1 + F_{cE}/F_c'}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c')}{2c} \right]^2 - \frac{F_{cE}/F_c'}{c}} \quad (616-14)$$

where:

F_c' = reference compression design value parallel to grain multiplied by all applicable adjustment factors except C_P (see 615.3), MPa

c = 0.8 for sawn lumber

= 0.85 for round timber poles and piles

= 0.9 for structural glued laminated timber, structural composite lumber, and cross-laminated timber.

616.7.1.6 For especially severe service conditions and/or extraordinary hazard, use of lower adjusted design values may be necessary.

616.7.2 Tapered Columns

For design of a column with rectangular cross section, tapered at one or both ends, the representative dimension, d_{rep} , for each face of the column shall be derived as follows:

$$d_{rep} = d_{min} + (d_{max} - d_{min}) \left[a - 0.15 \left(1 - \frac{d_{min}}{d_{max}} \right) \right] \quad (616-17)$$

where:

d_{rep} = representative dimension for tapered column, mm

d_{min} = the minimum dimension for that face of the column, mm

d_{max} = the maximum dimension for that face of the column, mm

Support Conditions

Large end fixed, small end unsupported or simply supported $a = 0.70$

Small end fixed, small end unsupported or simply supported $a = 0.30$

Both ends simply supported:

Tapered toward one end $a = 0.50$

Tapered toward both ends $a = 0.70$

For all other support conditions

$$d_{rep} = d_{min} + \frac{1}{3}(d_{max} - d_{min}) \quad (616-18)$$

Calculations of f_c and C_p shall be based on the representative dimension, d_{rep} . In addition, f_c at any cross section in the tapered column shall not exceed the reference compression design value parallel to grain multiplied by all applicable adjustment factors except the column stability factor, C_p .

616.7.3 Round Columns

The design of a column of round cross section shall be based on the design calculations for a square column of the same cross-sectional area and having the same degree of taper.

616.8 Tension Members

616.8.1 Tension Parallel to Grain

The actual tension stress or force parallel to grain shall be based on the net section area (see 616.1.2) and shall not exceed the adjusted tension design value.

616.8.2 Tension Perpendicular to Grain

Designs that induce tension stress perpendicular to grain shall be avoided whenever possible. When tension stress perpendicular to grain cannot be avoided, mechanical reinforcement sufficient to resist all such stresses shall be considered.

616.9 Combined Bending and Axial Loading

616.9.1 Bending and Axial Tension

Members subjected to a combination of bending and axial tension (see Figure 616.9-1) shall be so proportioned that:

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} \leq 1.0 \quad (616-19)$$

and

$$\frac{f_b - f_t}{F_b^{**}} \leq 1.0 \quad (616-20)$$

where:

F_b^* = reference bending design value multiplied by all applicable adjustment factors except C_L , MPa.

F_b^{**} = reference bending design value multiplied by all applicable adjustment factors except C_V , MPa.

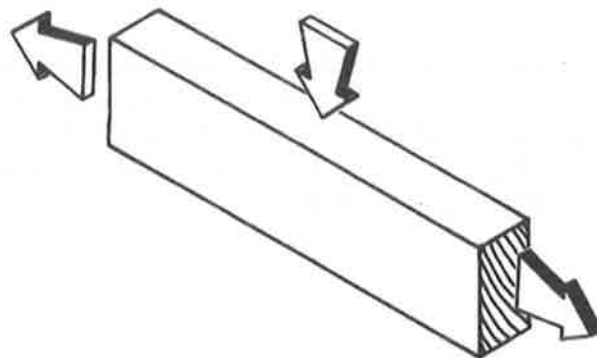


Figure 616.9-1 Combined Bending and Axial Tension

616.9.2 Bending and Axial Compression

Members subjected to a combination of bending about one or both principal axes and axial compression (see Figure 616.9-2) shall be so proportioned that:

$$\left[\frac{f_c}{F_c'} \right]^2 + \frac{f_{b1}}{F_{b1}' \left[1 - \frac{f_c}{F_{cE1}} \right]} + \frac{f_{b2}}{F_{b2}' \left[1 - \frac{f_c}{F_{cE2}} - \left(\frac{f_{b1}}{F_{bE}} \right)^2 \right]} \leq 1.0 \quad (616-21)$$

and

$$\frac{f_c}{F_{cE2}} + \left[\frac{f_{b1}}{F_{bE}} \right]^2 \leq 1 \quad (616-22)$$

where:

$$f_c < F_{cE1} = \frac{0.822 E_{min}'}{\left(\frac{\ell_{e1}}{d_1} \right)^2} \quad \text{for either uniaxial edgewise bending or biaxial bending}$$

and

$$f_c < F_{cE2} = \frac{0.822 E_{min}'}{\left(\frac{\ell_{e2}}{d_2} \right)^2} \quad \text{for uniaxial flatwise bending or biaxial bending}$$

and

$$f_{b1} < F_{bE} = \frac{1.20 E_{min}'}{(R_B)^2} \quad \text{for biaxial bending}$$

f_{b1} = actual edgewise bending stress (bending load applied to narrow face of member), MPa

f_{b2} = actual flatwise bending stress (bending load applied to wide face of member), MPa

d_1 = wide face dimension (see Figure 616.9-2), mm

d_2 = narrow face dimension (see Figure 616.9-2), mm

Effective column lengths, ℓ_{e1} and ℓ_{e2} , shall be determined in accordance with 616.7.1.2. F_c' , F_{cE1} , and F_{cE2} shall be determined in accordance with 615.3 and 616.7. F_{b1}' , F_{b2}' , and F_{bE} shall be determined in accordance with 615.3 and 616.3.3.

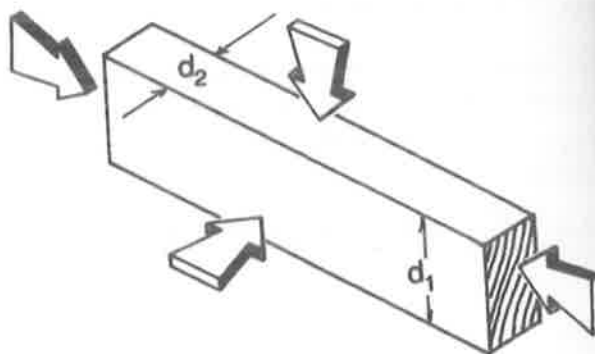


Figure 616.9-2 Combined Bending and Axial Tension

616.10 Design for Bearing

616.10.1 Bearing Parallel to Grain

616.10.1.1 The actual compressive bearing stress parallel to grain shall be based on the net bearing area and shall not exceed the reference compression design value parallel to grain multiplied by all applicable adjustment factors except the column stability factor, C_p .

616.10.1.2 F_c^* , the reference compression design values parallel to grain multiplied by all applicable adjustment factors except the column stability factor, applies to end-to-end bearing of compression members provided there is adequate lateral support and the end cuts are accurately squared and parallel.

616.10.1.3 When $f_c > 0.75 F_c^*$, bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogenous material with sufficient stiffness to distribute the applied load. Where a rigid insert is required for end-to-end bearing of compression members, it shall be equivalent to 1 mm a metal plate 1 mm thick or better, inserted with a snug fit between abutting ends.

616.10.2 Bearing Perpendicular to Grain

The actual compression stress perpendicular to grain shall be based on the net bearing area and shall not exceed the adjusted compression design value perpendicular to grain, $f_{c\perp} \leq F_{c\perp}'$. When calculating bearing area at the ends of bending members, no allowance shall be made for the fact that as the member bends, pressure upon the inner edge of the bearing is greater than at the member end.

616.10.3 Bearing at an Angle to Grain

The adjusted bearing design value at an angle to grain (see Figure 616.10-1) shall be calculated as follows:

$$F_{\theta}' = \frac{F_c^* F_{c\perp}'}{F_c^* \sin^2 \theta + F_{c\perp}' \cos^2 \theta} \quad (616.23)$$

where:

θ = angle between direction of load and direction of grain (longitudinal axis of member), degrees

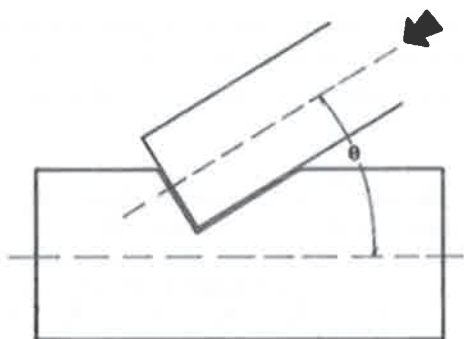


Figure 616.10-1 Bearing at an Angle to Grain

616.10.4 Bearing Area Factor, C_b

Reference compression design values perpendicular to grain, $F_{c\perp}$, apply to bearings of any length at the ends of a member, and to all bearings 150 mm or more in length at any other location. For bearings less than 150 mm in length and not nearer than 75 mm to the end of a member, the reference compression design value perpendicular to grain, $F_{c\perp}$, shall be permitted to be multiplied by the following bearing area factor, C_b :

$$C_b = \frac{\ell_b + 9.5}{\ell_b} \quad (616-24)$$

where:

ℓ_b = bearing length measured parallel to grain, mm

Equation 616-24 gives the following bearing area factors, C_b , for the indicated bearing length on such small areas as plates and washers.

The bearing length factors, C_b , for some values of ℓ_b are given in Table 616.10-1.

Table 616.10-1 Bearing Length Factor, C_b

ℓ_b (mm)	13	25	38	50	75	100	150 or more
C_b	1.73	1.38	1.25	1.19	1.13	1.10	1.00

For round bearing areas such as washers, the bearing length, ℓ_b , shall be equal to the diameter.

SECTION 617 SAWN LUMBER

617.1 General

617.1.1 Scope

Section 617 applies to engineering design with sawn lumber. Design procedures, reference design values, and other information herein apply only to lumber complying with the requirements specified below.

617.1.2 Identification of Lumber

When the reference design values specified herein are used, the lumber, including end-jointed or edge-glued lumber, shall be identified by the grade mark of, or certificate of inspection issued by, a lumber grading or inspection bureau or agency recognized as being competent.

617.1.3 Definitions

617.1.3.1 Structural sawn lumber consists of lumber classifications known as “Dimension”, “Beams and Stringers”, “Post and Timbers”, and “Decking”, with design values assigned to each grade.

617.1.3.2 “Dimension” refers to lumber from 50 mm to 100 mm (nominal) thick, and 50 mm (nominal) or more in width. Dimension lumber is further classified as Structural Light Framing, Light Framing, Studs, and Joists and Planks.

617.1.3.3 “Beams and Stringers” refers to lumber of rectangular cross section, 125 mm (nominal) or more thick, with width more than 50 mm greater than thickness, graded with respect to its strength in bending when loaded on the narrow face.

617.1.3.4 “Posts and Timbers” refers to lumber of square or approximately square cross section, 125 mm x 125 mm (nominal) and larger, with width not more than 50 mm greater than thickness, graded primarily for use as posts or columns carrying longitudinal load.

617.1.3.5 “Decking” refers to lumber from 50 mm to 100 mm (nominal) thick, tongued and grooved, or grooved for spline on the narrow face, and intended for use as a roof, floor, or wall membrane. Decking is graded for application in the flatwise direction, with the wide face of

the decking in contact with the supporting members, as normally installed.

617.1.4 Moisture Service Condition of Lumber

The reference design values for lumber specified herein are applicable to lumber that will be used under dry service conditions such as in most covered structures, where the moisture content (MC) in used will be a maximum of 19%, regardless of the moisture content at the time of manufacture. For lumber used under conditions where the moisture content of the wood in service will exceed 19% for an extended period of time, the design values shall be multiplied by the wet service factors, C_M , specified in Tables 617.1-1(a) and 617.1-1(b).

Table 617.1-1(a) Wet Service Factor, C_M
Visually Graded Sawn Lumber (MC > 19%)

	Strength Property					
	F_b	F_t	F_v	$F_{c\perp}$	F_c	E and E_{min}
Dimension Lumber	0.85 ¹	1.00	0.97	0.67	0.80 ²	0.90
Timbers	1.00	1.00	1.00	0.67	0.91	1.00
Decking	0.85 ²			0.67		0.90

¹ When $(F_b)(C_F) \leq 8 \text{ MPa}$, $C_M = 1.0$

² When $(F_c)(C_F) \leq 5 \text{ MPa}$, $C_M = 1.0$

Table 617.1-1(b) Wet Service Factor, C_M
Glued Laminated Timber (MC > 16%)

	Strength Property						
	F_b	F_t	F_v	$F_{c\perp}$	F_c	E and E_{min}	F_{rt}
All Species	0.80	0.80	0.875	0.53	0.73	0.833	0.875

617.1.5 Lumber Sizes

617.1.5.1 Lumber sizes referred to in this Specification are nominal sizes. Computations to determine the required sizes of members shall be based on the net dimensions (actual sizes) and not the nominal sizes.

617.1.5.2 For 100 mm (nominal) or thinner lumber, the net DRY dressed sizes shall be used in all computations of structural capacity regardless of the moisture content at the time of manufacture or use.

617.1.5.3 For 125 mm (nominal) and thicker lumber, the net GREEN dressed sizes shall be used in computations of structural capacity regardless of the moisture content at the time of manufacture or use.

617.1.5.4 Where a design is based on rough sizes or special sizes, the applicable moisture content and size used in design shall be clearly indicated in plans or specifications.

617.1.6 End-Jointed or Edge-Glued Lumber

Reference design values for sawn lumber are applicable to structural end-jointed or edge-glued lumber of the same species and grade. Such use shall include, but not be limited to light framing, studs, joists, planks, and decking.

617.1.7 Resawn or Remanufactured Lumber

617.1.7.1 When structural lumber is resawn or remanufactured, it shall be regraded, and reference design values for the regraded material shall apply.

617.1.7.2 When sawn lumber is cross cut to shorter lengths, the requirements of 617.1.7.1 shall not apply, except for reference bending design values for those Beam and Stringer grades where grading provisions for the middle 1/3 of the length of the piece differ from grading provisions for the outer thirds.

617.2 Reference Design Values

617.2.1 Reference Design Values

Reference design values for visually graded lumber are specified in Tables 615.2-1 and 615.2-2.

617.2.2 Other Species and Grades

Reference design values for species and grades of lumber not otherwise provided herein shall be established in accordance with appropriate ASTM standards and other technically sound criteria.

617.2.3 Basis for Reference Design Values

617.2.3.1 The reference design values in Tables 615.2-1 and 615.2-2 are for the design of structures where an individual member, such as a beam, girder, post or other

member, carries or is responsible for carrying its full design load. For repetitive member uses see 617.3.9.

617.2.3.2 Visually Graded Lumber. Reference design values for visually grade lumber in Tables 615.2-1 and 615.2-2 are based on the provisions of ASTM Standards D 245 and D 1990.

617.2.3.3 Machine Stress Rated (MSR) Lumber and Machine Evaluated Lumber (MEL). Reference design values for machine stress rated lumber and machine evaluated lumber in Tables 622.2-1 and 622.2-4 are determined by visual grading and nondestructive pretesting of individual pieces.

617.2.4 Modulus of Elasticity, E

617.2.4.1 Average Values. Reference design values for modulus of elasticity assigned to the visually graded species and grades of lumber listed in Tables 615.2-1 and 615.2-2 are average values which conform to ASTM Standards D 245 and D 1990. Adjustments in modulus of elasticity have been taken to reflect increases for seasoning, increases for density where applicable, and, where required, reductions have been made to account for the effect of grade upon stiffness. Reference modulus of elasticity design values are based upon the species or species group average in accordance with ASTM Standards D 1990 and D 2555.

617.2.4.2 Special Uses. Average reference modulus of elasticity design values listed in Tables 615.2-1 and 615.2-2 are to be used in design of repetitive member systems and in calculating the immediate deflection of single members which carry their full design load. In special applications where deflection is a critical factor, or where amount of deformation under long-term loading must be limited, the need for use of a reduced modulus of elasticity design value shall be determined.

617.2.5 Bending, F_b

617.2.5.1 Dimension Grades. Adjusted bending design values for Dimension grades apply to members with the load applied to either the narrow or wide face.

617.2.5.2 Decking Grades. Adjusted bending design values for Decking grades apply only when the load is applied to the wide face.

617.2.5.3 Post and Timber Grades. Adjusted bending design values for Post and Timber grades apply to members with the load applied to either the narrow or wide face.

617.2.5.4 Beam and Stringer Grades. Adjusted bending design values for Beam and Stringer Grades apply to members with the load applied to the narrow face. When Post and Timber sizes of lumber are graded to Beam and Stringer grade requirements, design values for the applicable Beam and Stringer grades shall be used. Such lumber shall be identified in accordance with 617.1.2.1 as conforming to Beam and Stringer grades.

617.2.5.5 Continuous or Cantilevered Beams. When Beams and Stringers are used as continuous or cantilevered beams, the design shall include a requirement that the grading provisions applicable to the middle 1/3 of the length shall be applied to at least the middle 2/3 of the length of pieces to be used as two span continuous beams, and to the entire length of pieces to be used over three or more spans or as cantilevered beams.

617.2.6 Compression Perpendicular to Grain, $F_{c\perp}$

For sawn lumber, the reference compression design values perpendicular to grain are based on a deformation limit that has been shown by experience to provide for adequate service in typical wood frame construction. The reference compression design values perpendicular to grain specified in Tables 615.2-1 and 615.2-2 are species group average values associated with a deformation level of 1 mm for a steel plate on wood member loading condition. One method for limiting deformation in special applications where it is critical, is use of a reduced compression design value perpendicular to grain. The following equation shall be used to calculate the compression design value perpendicular to grain for a reduced deformation level of 0.5 mm.

$$F_{c\perp 0.5} = 0.73 F_{c\perp} \quad (617-1)$$

where:

$F_{c\perp 0.5}$ = compression perpendicular to grain design value at 0.5 mm deformation limit, MPa

$F_{c\perp}$ = reference compression perpendicular to grain design value at 1 mm deformation limit (as tabulated in Tables 615.2-1 and 615.2-2), MPa

617.3 Adjustment of Reference Design Values

617.3.1 General

Reference Design values (F_b , F_t , F_v , $F_{c\perp}$, F_c , E , E_{min}) from Tables 615.2-1 and 615.2-2 shall be multiplied by the adjustment factors specified in Table 617.3-1 to determine adjusted design values (F_b' , F_t' , F_v' , $F_{c\perp}'$, F_c' , E' , E_{min}').

Table 617.3-1 Applicability of Adjustment Factors for Sawn Lumber

	Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Incising Factor	Repetitive Factor	Column Stability Factor	Buckling Stiffness Factor	Bearing Area Factor
$F_b' = F_b \times$	C_D	C_M	C_t	C_L	C_F	C_{fu}	C_i	C_r	-	-	-
$F_t' = F_t \times$	C_D	C_M	C_t	-	C_F	-	C_i	-	-	-	-
$F_v' = F_v \times$	C_D	C_M	C_t	-	-	-	C_i	-	-	-	-
$F_c' = F_c \times$	C_D	C_M	C_t	-	C_F	-	C_i	-	C_P	-	-
$F_{c\perp}' = F_{c\perp} \times$	-	C_M	C_t	-	-	-	C_i	-	-	-	C_b
$E' = E \times$	-	C_M	C_t	-	-	-	C_i	-	-	-	-
$E_{min}' = E_{min} \times$	-	C_M	C_t	-	-	-	C_i	-	-	C_T	-

where:

E_{min}' = Reference modulus of elasticity value multiplied by all applicable adjustment factors

E_{min} = Reference modulus of elasticity for beam and column stability

$$= E_{0.05}(1.03)/(1.66)$$

$E_{0.05}$ = Modulus of elasticity level exceeded by about 95% of the individual pieces

$$= E[1 - 1.645 COV_E]$$

E = Reference modulus of elasticity

COV_E = coefficient of variation in modulus of elasticity

= 0.25 for Visually graded sawn lumber

= 0.11 for Machine Stress Rate lumber

1.03 = adjustment factor to convert E values to a pure bending basis

1.66 = factor of safety

617.3.2 Load Duration Factor, C_D

All reference design values except modulus of elasticity, E , modulus of elasticity for beam and column stability, E_{min} , and compression perpendicular to grain, $F_{c\perp}$, shall be multiplied by load duration factors, C_D , as specified in 615.3.2.

617.3.3 Wet Service Factor, C_M

Reference design values for structural sawn lumber are based on the moisture service conditions specified in 617.1.4. When the moisture content of structural members in use differs from these moisture service conditions, reference design values shall be multiplied by the wet service factors, C_M , specified in Table 617.1-1(a).

617.3.4 Temperature Factor C_t

When structural members will experience sustained exposure to elevated temperatures up to 66°C, reference design values shall be multiplied by the temperature factors, C_t , specified in 615.3.3.

617.3.5 Beam Stability Factor, C_L

Reference bending design values, F_b , shall be multiplied by the beam stability factor, C_L , specified in 616.3.3.

617.3.6 Size Factor, C_F

617.3.6.1 Where the depth of a rectangular sawn lumber bending member 125 mm or thicker exceeds 300 mm, the reference bending design values, F_b , shall be multiplied by the following size factor:

$$C_F = \left(\frac{300}{d} \right)^{1/9} \quad (617-2)$$

where:

C_F = size factor

d = depth of beam, mm

617.3.6.2 For beams of circular cross section with a diameter greater than 340 mm, or for 300 mm or larger square beams loaded in the plane of the diagonal, the size factor shall be determined in accordance with 617.3.6.1 on the basis of an equivalent conventionally loaded square beam of the same cross-sectional area.

617.3.7 Flat Use Factor, C_{fu}

Bending design adjusted by size factors (C_F) are based on loads applied to narrow face. When sawn lumber is used flatwise, (i.e. load applied to wide face) bending design value, F_b , shall also be multiplied by the following flat-use factors:

Table 617.3-2 Visually-Graded Sawn Lumber
Flat-use Factors, C_{fu}

Width (depth)	Thickness (breadth)	
	50mm & 75mm	100mm
50 mm & 75 mm	1.0	-
100mm	1.1	1.0
125mm	1.1	1.05
150mm	1.15	1.05
200mm	1.15	1.05
250mm & wider	1.20	1.1

Table 617.3-3 Machine-Graded¹ Sawn Lumber
Flat-use Factors, C_{fu}

Width (depth)	Thickness (breadth)
	50mm
50mm & 75mm	1.0
100mm	1.1
125mm	1.1
150mm	1.15
200mm	1.15
250mm & wider	1.20

¹ Refer to Section 622

617.3.8 Incising Factor, C_i

Many species readily accept preservative treatments, while others don't. For species that are not easily treated, incising is used to make the treatment effective.

Some design values for sawn lumber must be adjusted if incising is used to increase the penetration of the preservatives. The incising factors, C_i , to be used shall be as follows:

Table 617.3-4 Incising Factor, C_i

	E	F_b	F_t	F_v	$F_{c\perp}$
C_i	0.95	0.80	0.80	0.80	1.0

For compression perpendicular to grain as well as for non-incised treated lumber, $C_i = 1.0$.

617.3.9 Repetitive Member Factor, C_r ,

A repetitive member system is defined as one that has:

1. Three or more parallel members of Dimension lumber or structural composite lumber;
2. Members spaced not more than 600 mm on center;
3. Members connected together by a load-distributing element such as roof, floor, or wall sheathing that has been designed or has been proven by experience to transmit the design load to adjacent members without displaying structural weakness or unacceptable deflection.

For a repetitive member system, the reference F_b may be multiplied by a repetitive member factor, $C_r = 1.15$. For all other framing systems, $C_r = 1.0$.

617.3.10 Column Stability Factor, C_P

Reference compression design values parallel to grain, F_c , shall be multiplied by the column stability factor, C_P , specified in 616.7.

617.3.11 Buckling Stiffness Factor, C_T

Reference modulus of elasticity for beam and column stability, E_{min} , shall be permitted to be multiplied by the buckling stiffness factor, C_T , as specified in 617.4.2.

617.3.12 Bearing Area Factor, C_b

Reference compression design values perpendicular to grain, $F_{c\perp}$, shall be permitted to be multiplied by the bearing area factor, C_b , as specified in 616.10.4.

617.3.13 Pressure-Preservative Treatment

Reference design values apply to sawn lumber pressure-treated by an approved process and preservative. Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with water-borne preservatives.

617.4 Special Design Considerations**617.4.1 Stability of Bending Members**

617.4.1.1 Sawn lumber bending members shall be designed in accordance with the lateral stability calculations in 616.3.3 or shall meet the lateral support requirements in 617.4.1.2 and 617.4.1.3.

617.4.1.2 As an alternative to 617.4.1.1, rectangular sawn lumber beams, rafters, joints, or other bending members, shall be designed in accordance with the following provisions to provide restraint against rotation or lateral displacement. If the depth to breadth, d/b , based on nominal dimensions is:

(a) $d/b \leq 2$; no lateral support shall be required.

(b) $2 < d/b \leq 4$; the ends shall be held in position, as by full depth solid blocking, bridging, hangers, nailing, or bolting to other framing members, or other acceptable means.

(c) $4 < d/b \leq 5$; the compression edge of the member shall be held in line for its entire length to prevent lateral displacement, as by adequate sheathing or subflooring, and ends at point of bearing shall be held in position to prevent rotation and/or lateral displacement.

(d) $5 < d/b \leq 6$; bridging, full depth solid blocking or diagonal cross bracing shall be installed at intervals not exceeding 2.40 m, the compression edge of the member shall be held in line as by adequate sheathing or subflooring, and the ends at points of bearing shall be held in position to prevent rotation and/or lateral displacement.

(e) $6 < d/b \leq 7$; both edges of the member shall be held in line for their entire length and ends at points of bearing shall be held in position to prevent rotation and/or lateral displacement.

617.4.1.3 If a bending member is subjected to both flexure and axial compression, the depth to breadth ratio shall be no more than 5 to 1 if one edge is firmly held in line. If under all combinations of load, the unbraced edge of the member is in tension, the depth to breadth ratio shall be no more than 6 to 1.

617.4.2 Wood Trusses

617.4.2.1 Increased chord stiffness relative to axial loads where a 50 mm x 100 mm or smaller sawn lumber truss compression chord is subjected to combined flexure and axial compression under dry service condition and has 9 mm or thicker plywood sheathing nailed to the narrow face of the chord in accordance with code required roof sheathing fastener schedules, shall be permitted to be accounted for by multiplying the reference modulus of elasticity design value for beam and column stability, E_{min} , by the buckling stiffness factor, C_T , in column stability calculations (see 616.7). When $\ell_e \leq 2.40 m$, C_T shall be calculated as follows:

$$C_T = 1 + \frac{K_m \ell_e}{K_T E} \quad (617-3)$$

where:

- ℓ_e = effective column length of truss compression chord, mm
- K_m = 0.62 for wood seasoned to 19% moisture content or less at the time of plywood attachment
= 0.33 for unseasoned or partially seasoned wood at the time of plywood attachment
- K_T = $1 - 1.645 COV_E$
= 0.59 for visually graded lumber
= 0.75 for machine evaluated lumber (MEL)
= 0.82 for products with $COV_E \leq 0.11$

When $\ell_e > 2.40 m$, C_T shall be calculated based on $\ell_e = 2.40 m$

617.4.3 Notches

617.4.3.1 End notches, located at the ends of sawn lumber bending members for bearing over a support, shall be permitted, and shall not exceed $\frac{1}{4}$ the beam depth (see Figure 617.4-1).

617.4.3.2 Interior notches, located in the outer thirds of the span of a single span sawn lumber bending member, shall be permitted, and shall not exceed $\frac{1}{6}$ the depth of the member. Interior notches on the tension side of 90 mm or greater thickness (100 mm nominal thickness) sawn lumber bending members are not permitted (see Figure 617.4-1).

617.4.3.3 See 616.1.2 and 616.4.3 for effect of notches on strength.

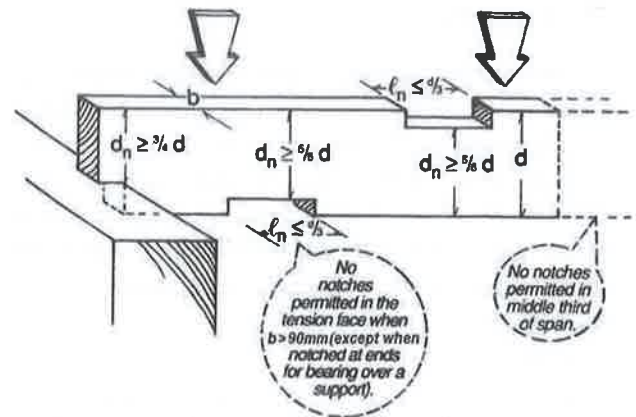


Figure 617.4-1 Notch Limitations for Sawn Lumber Beams

SECTION 618 STRUCTURAL GLUED LAMINATED TIMBER

618.1 General

618.1.1 Scope

618.1.1.1 Section 618 applies to engineering design with structural glued laminated timber. Basic requirements are provided in this Specification.

618.1.1.2 Design procedures, reference design values and other information provided herein apply only to structural glued laminated timber conforming to all pertinent provisions of the specifications.

618.1.2 Definition

The term “structural glued laminated timber” refers to an engineered, stress rated product of a timber laminating plant, comprising assemblies of specially selected and prepared wood laminations bonded together with adhesives. The grain of all laminations is approximately parallel longitudinally. The separate laminations shall not exceed 50 mm in net thickness and are permitted to be comprised of:

- one piece
- pieces joined end-to-end to form any length
- pieces placed or glued edge-to-edge to make wider ones
- pieces bent to curved form during gluing.

618.1.3 Standard Sizes

618.1.3.1 Standard finished widths of structural glued laminated members shall meet the size requirements of a design or other special requirements.

618.1.3.2 The length and net dimensions of all members shall be specified. Additional dimensions necessary to define non-prismatic members shall be specified.

618.1.4 Service Conditions

618.1.4.1 Reference design values for dry service conditions shall apply when the moisture content in service is less than 16%, as in most covered structures.

618.1.4.2 Reference design values for glued laminated timber shall be multiplied by the wet service factors, C_M ,

specified in Tables 617.1-1(b) when the moisture content in service is 16% or greater, as may occur in exterior or submerged construction, or humid environments.

618.2 Reference Design Values

618.2.1 Reference Design Values

Reference design values for softwood and hardwood structural glued laminated timber shall be based on other substantiated information from an approved source.

618.2.2 Orientation of Member

Reference design values for structural glued laminated timber are dependent on the orientation of the laminations relative to the applied loads. Subscripts are used to indicate design values corresponding to a given orientation. The orientations of the cross-sectional axes for structural glued laminated timber are shown in Figure 618.2-1. The x-x axis runs parallel to the wide face of the laminations. The y-y axis runs perpendicular to the wide face of the laminations.

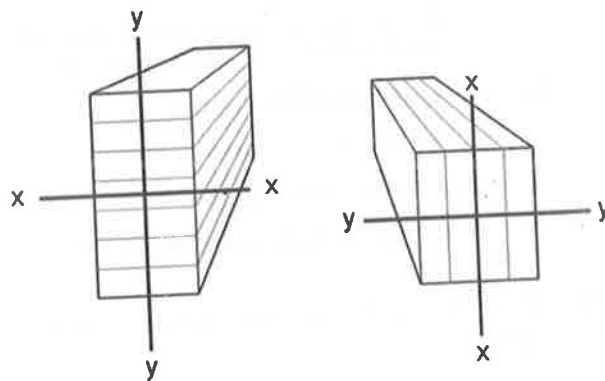


Figure 618.2-1 Axis Orientations

618.2.3 Balanced and Unbalanced Layups

Structural glued laminated timbers are permitted to be assembled with laminations of the same lumber grades placed symmetrically or asymmetrically about the neutral axis of the member. Symmetrical layups are referred to as “balanced” and have the same design values for positive and negative bending. Asymmetrical layups are referred to as “unbalanced” and have lower design values for negative bending than for positive bending. The top side of unbalanced members is required to be marked “TOP” by the manufacturer.

618.2.4 Bending, F_{bx+} , F_{bx-} , F_{by}

The reference bending design values F_{bx+} and F_{bx-} , shall apply to members with loads causing bending about the x-x axis. The reference bending design value for positive bending, F_{bx+} , shall apply for bending stresses causing tension at the bottom of the beam. The reference bending design value for negative bending, F_{bx-} , shall apply for bending stresses causing tension at the top of the beam. The reference bending design values, F_{by} , shall apply to members with loads causing bending about the y-y axis.

618.2.5 Compression Perpendicular to Grain, $F_{c\perp x}$, $F_{c\perp y}$

The reference compression design value perpendicular to grain, $F_{c\perp x}$, shall apply to members with bearing loads on the wide faces of the laminations.

The reference compression design value perpendicular to grain, $F_{c\perp y}$, shall apply to members with bearing loads on the narrow edges of the laminations.

The reference compression design values perpendicular to grain are based on a deformation limit of 1 mm obtained from testing in accordance with ASTM D143. The compression perpendicular to grain stress associated with a 0.5 mm deformation limit shall be permitted to be calculated as 73% of the reference value (See also 617.2.6).

618.2.6 Shear Parallel to Grain, F_{vx} , F_{vy}

The reference shear design value parallel to grain, F_{vx} shall apply to members with shear loads causing bending about the x-x axis. The reference shear design value parallel to grain, F_{vy} , shall apply to members with shear loads causing bending about the y-y axis.

The reference shear design values parallel to grain shall apply to prismatic members except those subject to impact or repetitive cyclic loads. For non-prismatic members and for all members subject to impact or repetitive cyclic loads, the reference shear design values parallel to grain shall be multiplied by the shear reduction factor specified in 618.3.10. This reduction shall also apply to the design of connections transferring loads through mechanical fasteners (see 616.4.3).

Prismatic members shall be defined as straight or cambered members with constant cross-section. Non-

prismatic members include, but not limited to: arches, tapered beams, curved beams, and notched members.

The reference shear design value parallel to grain, F_{vy} , is tabulated for members with four or more laminations. For members with two or three laminations, the reference design value shall be multiplied by 0.84 or 0.95, respectively.

618.2.7 Modulus of Elasticity, E_x , $E_{x\min}$, E_y , $E_{y\min}$

The reference modulus of elasticity, E_x , shall be used for determination of deflections due to bending about the x-x axis.

The reference modulus of elasticity, $E_{x\min}$, shall be used for beam and column stability calculations for members buckling about the x-x axis.

The reference modulus of elasticity, E_y , shall be used for determination of deflections due to bending about the y-y axis.

The reference modulus of elasticity, $E_{y\min}$, shall be used for beam and column stability calculations for members buckling about the y-y axis.

For the calculation of extensional deformations, the axial modulus of elasticity shall be permitted to be estimated as $E_{axial} = 1.05 E_y$.

618.2.8 Radial Tension, F_{rt}

For curved bending members, the radial tension design values perpendicular to grain, F_{rt} , shall be based on other substantiated information from an approved source.

618.2.9 Radial Compression, F_{rc}

For curved bending members, the reference radial compression design value, F_{rc} , shall be taken as the reference compression perpendicular to grain design value on the side face, $F_{c\perp y}$.

618.2.10 Other Species and Grades

Reference design values for species and grades of structural glued laminated timber not otherwise provided herein shall be based on other substantiated information from an approved source.

618.3 Adjustment of Reference Design Values

618.3.1 General

Reference Design values (F_b , F_t , F_v , F_c , F_{rt} , E , E_{min}) shall be multiplied by the adjustment factors specified in Table 618.3-1 to determine adjusted design values (F_b' , F_t' , F_v' , F_c' , F_{rt}' , E' , E_{min}').

Table 618.3-1 Applicability of Adjustment Factors for Structural Glued Laminated Timber

	Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor ¹	Volume Factor ¹	Flat Use Factor	Curvature Factor	Stress Interaction Factor	Shear Reduction Factor	Column Stability Factor	Bearing Area Factor
$F_b' = F_b \times C_D C_M C_t C_L C_V C_{fu} C_c C_i$	C_D	C_M	C_t	C_L	C_V	C_{fu}	C_c	C_i	-	-	-
$F_t' = F_t \times C_D C_M C_t$	C_D	C_M	C_t	-	-	-	-	-	-	-	-
$F_v' = F_v \times C_D C_M C_t$	C_D	C_M	C_t	-	-	-	-	-	C_V	-	-
$F_{rt}' = F_{rt} \times C_D C_M C_t$	C_D	C_M	C_t	-	-	-	-	-	-	-	-
$F_c' = F_c \times C_D C_M C_t$	C_D	C_M	C_t	-	-	-	-	-	-	C_P	-
$F_{c\perp}' = F_{c\perp} \times C_M C_t$	-	C_M	C_t	-	-	-	-	-	-	-	C_b
$E' = E \times C_M C_t$	-	C_M	C_t	-	-	-	-	-	-	-	-
$E_{min}' = E_{min} \times C_M C_t$	-	C_M	C_t	-	-	-	-	-	-	-	-

¹ The beam stability factor, C_L , shall not apply simultaneously with the volume factor, C_V , for structural glued laminated timber bending members (see 618.3.6). Therefore, the lesser of these adjustment factors apply.

where:

- E_{min}' = Reference modulus of elasticity value multiplied by all applicable adjustment factors
- E_{min} = Reference modulus of elasticity for beam and column stability
 $= E_{0.05}(1.03)/(1.66)$
- $E_{0.05}$ = Modulus of elasticity level exceeded by about 95% of the individual pieces
 $= E[1 - 1.645COV_E]$
- E = Reference modulus of elasticity
- COV_E = coefficient of variation in modulus of elasticity
 $= 0.10$ for Structural Glued Laminated timber
- 1.03 = adjustment factor to convert E values to a pure bending basis
- 1.66 = factor of safety

618.3.2 Load Duration Factor, C_D

All reference design values except modulus of elasticity, E , modulus of elasticity for beam and column stability, E_{min} , and compression perpendicular to grain, F_c , shall be multiplied by load duration factors, C_D , as specified in 616.3.2.

618.3.3 Wet Service Factor, C_M

Reference design values for structural glued laminated timber are based on the moisture service conditions specified in 618.1.4. When the moisture content of structural members in sue differs from these moisture service conditions, reference design values shall be multiplied by the wet service factors, C_M , specified in Table 617.1-1(b).

618.3.4 Temperature Factor, C_t

When structural members will experience sustained exposure to elevated temperatures up to 66°C, reference design values shall be multiplied by the temperature factors, C_t , specified in 615.3.3.

618.3.5 Beam Stability Factor, C_L

Reference bending design values, F_b , shall be multiplied by the beam stability factor, C_L , specified in 616.3.3. The beam stability factor, C_L , shall not apply simultaneously with the volume factor, C_V , for structural glued laminated timber bending members (see 618.3.6). Therefore, the lesser of these adjustment factors shall apply.

618.3.6 Volume Factor, C_V

When structural glued laminated timber members are loaded in bending about the x-x axis, the reference bending design values, F_{bx+} , and F_{bx-} , shall be multiplied by the following volume factor:

$$C_V = \left(\frac{6.4}{L}\right)^{\frac{1}{x}} \left(\frac{300}{d}\right)^{\frac{1}{x}} \left(\frac{130}{b}\right)^{\frac{1}{x}} \leq 1.0 \quad (618-1)$$

where:

- L = length of bending member between points of zero moment, m
- d = depth of bending member, mm
- b = width (breadth) of bending member. For multiple piece width layouts, b = width of widest piece used in the layout. Thus, $b \leq 275$ mm.
- x = 10 if specific value has not been established for the species used

The volume factor, C_V , shall not apply simultaneously with the beam stability factor, C_L (see 616.3.3). Therefore, the lesser of these adjustment factors shall apply.

618.3.7 Flat Use Factor, C_{fu}

When structural glued laminated timber is loaded in bending about the y-y axis and the member dimension parallel to the wide face of the laminations, d_y (see Figure 618.3-1), is less than 300 mm, the reference bending design value, F_{by} , shall be permitted to be multiplied by the flat use factor, C_{fu} , calculated by the following formula:

$$C_{fu} = \left(\frac{300}{d_y} \right)^{1/9} \quad (618-2)$$

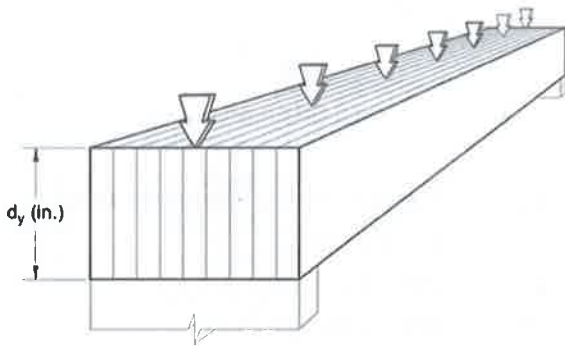


Figure 618.3-1 Depth, d_y , for Flat Use Factor

618.3.8 Curvature Factor, C_c

For curved portions of bending members, the reference bending design value shall be multiplied by the following curvature factor:

$$C_c = 1 - 2000 \left(\frac{t}{R} \right)^2 \quad (618-3)$$

where:

- t = thickness of laminations, mm
- R = radius of curvature of inside face of member, mm
- $\frac{t}{R} \leq \frac{1}{100}$ for hardwoods
- $\frac{t}{R} \leq \frac{1}{125}$ for softwoods

The curvature factor shall not apply to reference design values in the straight portion of a member, regardless of curvature elsewhere.

618.3.9 Stress Interaction Factor, C_I

For the tapered portion of bending members tapered on the compression face, the reference bending design value, F_{bx} , shall be multiplied by the following stress interaction factor:

$$C_I = \frac{1}{\sqrt{1 + \left(\frac{F_b \tan \theta}{F_v C_{vr}} \right)^2 + \left(\frac{F_b \tan^2 \theta}{F_{c\perp}} \right)^2}} \quad (618-4)$$

where:

θ = angle of taper, degrees

For members tapered on the compression face, the stress interaction factor, C_I , shall not apply simultaneously with the volume factor, C_V , therefore, the lesser of these adjustment factors shall apply.

For the tapered portion of the bending members tapered on the tension face, the reference bending design value, F_{bx} , shall be multiplied by the following stress interaction factor:

$$C_I = \frac{1}{\sqrt{1 + \left(\frac{F_b \tan \theta}{F_v C_{vr}} \right)^2 + \left(\frac{F_b \tan^2 \theta}{F_{rt}} \right)^2}} \quad (618-5)$$

where:

θ = angle of taper, degrees

For members tapered on the tension face, the stress interaction factor, C_I , shall not apply simultaneously with the beam stability factor, C_L , therefore, the lesser of these adjustment factors shall apply.

Taper cuts on the tension face of structural glued laminated timber beams are not recommended.

618.3.10 Shear Reduction Factor, C_{vr}

The reference shear design values, F_{vx} and F_{vy} , shall be multiplied by the shear reduction factor, $C_{vr} = 0.72$ where any of the following conditions apply:

1. Design of non-prismatic members.
2. Design of members subject to impact or repetitive cyclic loading.
3. Design of members at notches (616.4.3.2).
4. Design of members at connections (616.4.3.3).

618.3.11 Column Stability Factor, C_P

Reference compression design values parallel to grain, F_c , shall be multiplied by the column stability factor, C_P , specified in 616.7.

618.3.12 Bearing Area Factor, C_b

Reference compression design values perpendicular to grain, $F_{c\perp}$, shall be permitted to be multiplied by the bearing area factor, C_b , as specified in 616.10.4.

618.3.13 Pressure-Preservative Treatment

Reference design values apply to structural glued laminated timber treated by an approved process and preservative. Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with water-borne preservatives.

618.4 Special Design Considerations**618.4.1 Curved Bending Members with Constant Cross Section**

618.4.1.1 Curved bending members with constant rectangular cross section shall be designed for flexural strength in accordance with 616.3.

618.4.1.2 Curved bending members with constant rectangular cross section shall be designed for shear strength in accordance with 616.4, except that the provisions of 616.4.3.1 shall not apply. The shear reduction factor from 618.3.10 shall apply.

618.4.1.3 The radial stress induced by a bending moment in a curved bending member of constant rectangular cross section is:

$$f_r = \frac{3M}{2Rbd} \quad (618-6)$$

where:

- M = bending moment, N·m
 R = radius of curvature at center line of member, mm

Where the bending moment is in the direction tending to decrease curvature (increase the radius), the radial stress shall not exceed the adjusted radial tension design value perpendicular to the grain, $f_r \leq F_{rt}'$, unless mechanical

reinforcing sufficient to resist all radial stresses is used. In no case shall f_r exceed $(1/3)F_v'$.

Where the bending moment is in the direction tending to increase curvature (decrease the radius), the radial stress shall not exceed the adjusted radial compressional design $f_r \leq F_{rc}'$.

618.4.1.4 The deflection of curved bending members with constant cross section shall be determined in accordance with 616.5. Horizontal displacements at the supports shall also be considered.

618.4.2 Double-Tapered Curved Bending Members

618.4.2.1 The bending stress induced by a bending moment, M , at the peaked section of a double-tapered curved bending member (see Figure 618.4-1) shall be calculated as follows:

$$f_b = K_\phi \frac{6M}{bd_c^2} \quad (618-7)$$

where:

- K_ϕ = empirical bending stress shape factor
 $= 1 + 2.7 \tan \phi_T$
 ϕ_T = angle of roof slope, degrees
 M = bending moment, N·m
 d_c = depth at peaked section of member, mm

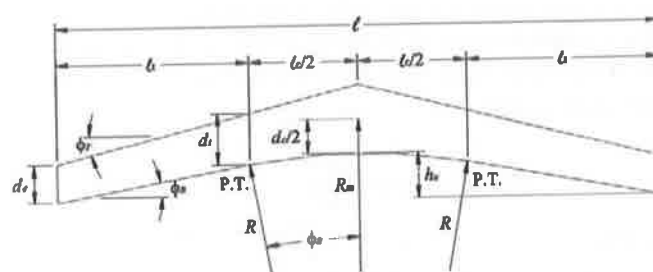


Figure 618.4-1 Double-Tapered Curved Bending Member

The stress interaction factor from 618.3.9 shall apply for flexural design in the straight-tapered segments of double-tapered curved bending members.

618.4.2.2 Double-tapered curved members shall be designed for shear strength in accordance with 616.4, except that the provisions of 616.4.3.1 shall not apply. The shear reduction factor from 618.3.10 shall apply.

618.4.2.3 The radial stress induced by bending moment in a double-tapered curved member shall be calculated as follows:

$$f_r = K_{rs} C_{rs} \frac{6M}{bd_c^2} \quad (618-8)$$

where:

- K_{rs} = empirical radial stress factor
= $0.29(d_c/R_m) + 0.32 \tan^{1.2} \phi_T$
- C_{rs} = empirical load-shape radial stress reduction factor
= $0.27 \ln(\tan \phi_T) + 0.28 \ln(\ell/\ell_c) - 0.8(d_c/R_m) + 1 \leq 1.0$ for uniformly loaded members where $d_c/R_m \leq 0.3$
= 1.0 for members subject to constant moment
- ℓ = span length, mm
- ℓ_c = length between tangent points, mm
- M = bending moment, N·m
- d_c = depth at peaked section of member, mm
- R_m = radius of curvature at center line of member, mm
= $R + d_c/2$
- R = radius of curvature of inside face of member, mm

Where the bending moment is in the direction tending to decrease curvature (increase the radius), the radial stress shall not exceed the adjusted radial tension design value perpendicular to grain, $f_r \leq F_{rt}$, unless mechanical reinforcing sufficient to resist all radial stresses is used. In no case shall f_r exceed $(1/3)F_{vx}'$.

Where the bending moment is in the direction tending to increase curvature (decrease the radius), the radial stress shall not exceed the adjusted radial compression design value, $f_r \leq F_{rc}'$.

618.4.2.4 The deflection of double-tapered curved members shall be determined in accordance with 616.5, except that the mid-span deflection of a symmetrical double-tapered curved beam subject to uniform loads shall be permitted to be calculated by the following empirical formula:

$$\Delta_c = \frac{5\omega\ell^4}{32E_x b(d_{equiv})^3} \quad (618-9)$$

where:

- Δ_c = vertical deflection at midspan, mm
- ω = uniformly distributed load, N/mm
- $d_{equiv} = (d_e + d_c)(0.5 + 0.735 \tan \phi_T) - 1.41 d_c \tan \phi_B$
- d_e = depth at the ends of the member, mm
- d_c = depth at the peaked section of member, mm
- ϕ_T = angle of roof slope, degrees
- ϕ_B = soffit slope at the ends of the member, degrees

The horizontal deflection at the supports of symmetrical double-tapered curved beams shall be permitted to be estimated as

$$\Delta_H = \frac{2h\Delta_c}{\ell} \quad (618-10)$$

where:

- Δ_H = horizontal deflection at either support, mm
- $h = h_a - d_c/2 - d_e/2$
- $h_a = (\ell/2) \tan \phi_T + d_e$

618.4.3 Lateral Stability for Tudor Arches

The ratio of tangent point depth to breadth (d/b) of tudor arches (see Figure 618.4-2) shall not exceed 6, based on actual dimensions, when one edge of the arch is braced by decking fastened directly to the arch, or braced at frequent intervals as by girts or roof purlins. Where such lateral bracing is not present, d/b shall not exceed 5. Arches shall be designed for lateral stability in accordance with the provisions of 616.7 and 616.9.2.

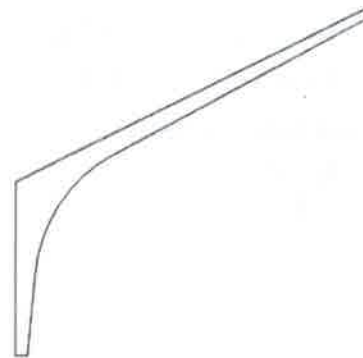


Figure 618.4-2 Tudor Arch

618.4.4 Tapered Straight Bending Members

618.4.4.1 Tapered straight beams (see Figure 618.4-3) shall be designed for flexural strength in accordance with 616.6. The stress interaction factor from 618.3.9 shall apply. For field-tapered members, the reference bending design value, F_{bx} , and the reference modulus of elasticity, E_x , shall be reduced according to the manufacturer's recommendations to account for the removal of high grade material near the surface of the member.

618.4.4.2 Tapered straight beams shall be designed for shear strength in accordance with 616.4, except that the provisions of 616.4.3.1 shall not apply. The shear reduction factor from 618.3.10 shall apply.

618.4.4.3 The deflection of tapered straight beams shall be determined in accordance with 616.5, except that the maximum deflection of a tapered straight beam subject to uniform loads shall be permitted to be calculated as equivalent to the depth, d_{equiv} , of an equivalent prismatic member of the same width where:

$$d_{equiv} = C_{dt} d_e \quad (618-11)$$

where:

- d_e = depth at the small end of the member, mm
 C_{dt} = empirical constant derived from relationship of equations for deflection of tapered straight beams and prismatic beams.

For symmetrical double-tapered beams:

$$C_{dt} = 1 + 0.66C_y \quad \text{when } 0 < C_y \leq 1$$

$$C_{dt} = 1 + 0.62C_y \quad \text{when } 1 < C_y \leq 3$$

For single-tapered beams:

$$C_{dt} = 1 + 0.46C_y \quad \text{when } 0 < C_y \leq 1.1$$

$$C_{dt} = 1 + 0.43C_y \quad \text{when } 1.1 < C_y \leq 2$$

For single- and double-tapered beams:

$$C_y = \frac{d_c - d_e}{d_e}$$

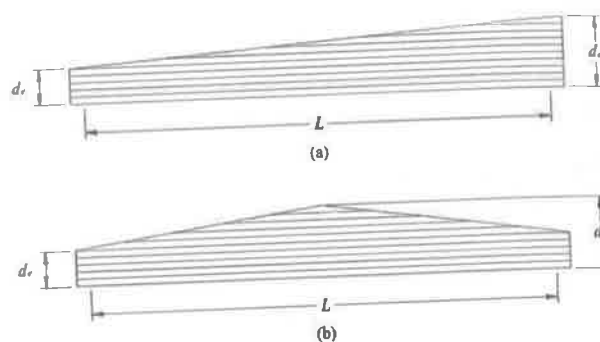


Figure 618.4-3 Tapered Straight Bending Members

618.4.5 Notches

618.4.5.1 The tension side of structural glued laminated timber bending members shall not be notched, except at ends of members for bearing over a support, and notch depth shall not exceed the lesser of 1/10 the depth of the member or 75 mm.

618.4.5.2 The compression side of structural glued laminated timber bending members shall not be notched, except at ends of members, and the notch depth on the compression side shall not exceed 2/5 the depth of the member. Compression side end-notches shall not extend into the middle 1/3 of the span.

Exception:

A taper cut on the compression edge at the end of a structural glued laminated timber bending member shall not exceed 2/3 the depth of the member and the length shall not exceed three times the depth of the member, 3d. For tapered beams where the taper extends into the middle 1/3 of the span, design shall be in accordance with 618.4.4.

618.4.5.3 Notches shall not be permitted on both the tension and compression face at the same cross-section

618.4.5.4 See 616.1.2 and 616.4.3 for the effect of notches on strength. The shear reduction factor from 618.3.10 shall apply for the evaluation of members at notches.

SECTION 619

TIMBER CONNECTORS AND FASTENERS

619.1 General

Timber connectors and fasteners may be used to transmit forces between wood members and between wood and metal members. The allowable loads and installation of timber connectors and fasteners shall be in accordance with the tables as provided in this Chapter. The allowable loads and installation of timber connectors shall be as set forth in Tables 619.1-1, 619.1-2, 619.1-3 and 619.1-4.

Safe loads and design practices for types of connectors and fasteners not mentioned or fully covered may be determined in a manner permitted by the Building Official.

619.2 Bolts

Safe loads in kN for bolts in shear in seasoned lumber shall not exceed the values set forth in Table 619.1-1.

Allowable shear values used to connect a wood to concrete or masonry are permitted to be determined as one half the tabulated double shear values for a wood member twice the thickness of the member attached to the concrete or masonry.

Table 619.1-1 Grouping of Species for Determining Allowable Loads for Timber Joints

I		II		III		IV	
Species	Relative Density	Species	Relative Density	Species	Relative Density	Species	Relative Density
Malabayabas	0.90	Makaasim	0.74	Malugai	0.61	Lingo-lingo	0.48
Sasalit	0.90	Kamagong	0.72	Dangakalan	0.58	Raintree	0.48
Agoho	0.84	Guijo	0.70	Apitong	0.57	Bayok	0.44
Liusin	0.79	Binggas	0.70	Salakin	0.56	Almaciga	0.42
Yakal	0.76	Katmon	0.68	Pine	0.55	Manggasinoro	0.42
Narig	0.72	Gatasan	0.67	Lanutan-bagyo	0.53	Yemane	0.42
Manggachapui	0.71	Bok-bok	0.64	Miau	0.52		
Molave	0.69	Kamatog	0.64	Palosapis	0.52		
		Lomarau	0.64	Malasaging	0.51		
		Kato	0.59	Vidal Lanutan	0.50		
		Pahutan	0.55	Gisau	0.50		
		Mahogany, big leaf	0.54	Nato	0.49		
		Antipolo	0.52	Bagtikan	0.44		
		Narra	0.52	Malaanonang	0.41		
		Malakauayan	0.50	Lauan	0.40		
		Dao	0.48				

Table 619.1-2 Reference Values on One Bolt (Double Shear) in Seasoned Wood Loaded at Both Ends

Length of Bolt in Main Member <i>L</i> (mm)	Diameter of Bolt <i>d</i> (mm)	Loaded parallel to grain (0°)				Loaded perpendicular to grain (90°)			
		Design value, <i>P</i> , per bolt, kN				Design value, <i>Q</i> , per bolt, kN			
		Species Group				Species Group			
		I	II	III	IV	I	II	III	IV
40	12	7.08	4.86	4.20	3.26	3.76	2.07	1.98	1.82
	16	8.75	6.01	5.20	4.01	4.19	2.30	2.21	2.02
	20	11.0	7.55	6.53	5.02	4.85	2.67	2.56	2.35
	22	12.1	8.30	7.19	5.52	5.04	2.77	2.66	2.43
	25	13.7	9.43	8.16	6.27	5.47	3.01	2.89	2.64
50	12	8.38	5.75	4.98	3.99	4.70	2.59	2.48	2.27
	16	10.8	7.42	6.43	5.00	5.23	2.88	2.76	2.53
	20	13.7	9.38	8.12	6.27	6.07	3.34	3.20	2.93
	22	15.1	10.3	8.95	6.90	6.30	3.46	3.32	3.04
	25	17.2	11.8	10.2	7.84	6.83	3.76	3.61	3.30
65	12	9.29	6.38	5.52	4.68	6.11	3.36	3.23	2.95
	16	13.0	8.95	7.74	6.29	6.80	3.74	3.59	3.29
	20	17.4	11.9	10.3	8.10	7.89	4.34	4.16	3.84
	22	19.3	13.2	11.4	8.97	8.19	4.50	4.32	3.96
	25	22.2	15.2	13.2	10.2	8.88	4.89	4.69	4.29
75	12	9.35	6.42	5.56	4.79	6.84	3.76	3.61	3.30
	16	13.8	9.48	8.21	6.85	7.85	4.32	4.14	3.79
	20	19.4	13.3	11.5	9.22	9.10	5.01	4.81	4.40
	22	21.9	15.0	13.0	10.2	9.45	5.20	4.99	4.56
	25	25.5	17.5	15.2	11.8	10.2	5.64	5.41	4.95
80	12	9.41	6.45	5.59	4.84	7.14	3.93	3.77	3.45
	16	14.1	9.66	8.36	7.09	8.37	4.61	4.42	4.05
	20	20.3	14.0	12.1	9.77	9.71	5.34	5.13	4.69
	22	22.9	15.7	13.6	10.9	10.1	5.54	5.32	4.87
	25	26.9	18.4	16.0	12.5	10.9	6.01	5.77	5.28
90	12	9.39	6.44	5.58	4.84	7.42	4.08	3.92	3.59
	16	14.3	9.79	8.48	7.28	9.20	5.06	4.86	4.45
	20	21.3	14.6	12.7	10.5	10.9	6.04	5.77	5.28
	22	24.9	17.1	14.8	12.0	11.3	6.24	5.98	5.48
	25	29.4	20.2	17.5	13.9	12.3	6.77	6.49	5.94
100	12	9.40	6.45	5.58	4.84	7.40	4.07	3.91	3.58
	16	14.2	9.78	8.47	7.34	9.84	5.41	5.19	4.76
	20	22.0	15.1	13.1	11.1	12.1	6.68	6.41	5.87
	22	25.9	17.8	15.4	12.8	12.6	7.35	6.65	6.45
	25	31.8	21.8	18.9	15.3	13.7	7.52	7.21	6.60
125	12	9.38	6.44	5.57	4.83	6.92	3.81	3.65	3.34
	16	14.2	9.77	8.46	7.33	10.1	5.56	5.33	4.89
	20	22.3	15.3	13.2	11.5	14.3	7.85	7.53	6.89
	22	26.9	18.5	16.0	13.8	15.3	8.44	8.10	7.42
	25	34.4	23.6	20.4	17.3	17.1	9.40	9.02	8.26
	28	41.8	28.7	24.8	20.5	18.5	10.2	9.78	8.95

¹ Tabulated design values apply to ONE bolt in double shear

² To obtain adjusted lateral design values (*P'*, *Q'*), tabulated lateral design values (*P*, *Q*) for bolts shall be multiplied to all applicable adjustment factors based on National Design Specification for Wood Construction (NDS) 2015 Edition

³ When loaded at an angle θ to grain, lateral design value shall be computed using $N' = \frac{P'Q'}{P' \sin^2 \theta + Q' \cos^2 \theta}$

Table 619.1-2 (cont'd) Reference Values on One Bolt (Double Shear) in Seasoned Wood Loaded at Both Ends

Length of Bolt in Main Member L (mm)	Diameter of Bolt d (mm)	Loaded parallel to grain (0°)				Loaded perpendicular to grain (90°)			
		Design value, P , per bolt, kN				Design value, Q , per bolt, kN			
		Species Group				Species Group			
		I	II	III	IV	I	II	III	IV
140	12	9.38	6.44	5.57	4.83	6.92	3.81	3.65	3.34
	16	14.2	9.77	8.46	7.33	10.1	5.56	5.33	4.89
	20	22.3	15.3	13.2	11.5	14.3	7.85	7.53	6.89
	22	26.9	18.5	16.0	13.8	15.3	8.44	8.10	7.42
	25	34.4	23.6	20.4	17.3	17.1	9.40	9.02	8.26
	28	41.8	28.7	24.8	20.5	18.5	10.2	9.78	8.95
150	16	14.2	9.76	8.45	7.33	9.59	5.28	5.06	4.64
	20	22.3	15.3	13.2	11.5	14.7	8.11	7.78	7.13
	22	26.9	18.5	16.0	13.8	16.7	9.21	8.84	8.09
	25	34.6	23.8	20.6	17.8	19.7	10.9	10.4	9.54
	28	43.5	29.9	25.9	22.1	21.9	12.1	11.6	10.6
180	16	14.2	9.72	8.42	7.32	8.84	4.86	4.66	4.27
	20	22.2	15.2	13.2	11.4	14.1	7.76	7.45	6.82
	22	26.9	18.5	16.0	13.9	16.6	9.12	8.75	8.01
	25	34.8	23.9	20.6	17.9	20.8	11.4	11.0	10.0
	28	43.7	30.0	25.9	22.5	24.6	13.5	13.0	11.9
190	16	14.2	9.75	8.44	7.30	8.59	4.73	4.54	4.15
	20	22.2	15.3	13.2	11.4	13.8	7.61	7.30	6.69
	22	26.9	18.5	16.0	13.9	16.3	8.99	8.63	7.90
	25	34.8	23.9	20.7	17.9	20.7	11.4	10.9	10.0
	28	43.6	29.9	25.9	22.5	25.0	13.8	13.2	12.1
200	16	14.2	9.90	8.44	7.30	8.37	4.61	4.42	4.05
	20	22.2	15.2	13.2	11.4	13.5	7.40	7.13	6.50
	22	26.8	18.4	16.0	13.9	16.1	8.84	8.48	7.77
	25	34.6	23.8	20.6	17.8	20.5	11.3	10.8	9.91
	28	43.6	29.9	25.9	22.4	25.3	13.9	13.3	12.2
230	20	22.2	15.2	13.2	11.4	12.7	6.97	6.69	6.12
	22	26.8	18.4	15.9	13.9	15.1	8.32	7.98	7.31
	25	34.7	23.8	20.6	17.9	19.7	10.9	10.4	9.54
	28	43.5	29.9	25.9	22.5	24.8	13.7	13.1	12.0
	32	56.8	39.0	33.8	29.3	31.9	17.5	16.8	15.4
240	20	22.2	15.2	13.2	11.4	12.4	6.81	6.54	5.98
	22	26.8	18.4	16.0	13.8	14.8	8.15	7.83	7.16
	25	34.7	23.8	20.6	17.9	19.4	10.7	10.2	9.37
	28	43.6	29.9	25.9	22.5	24.6	13.5	13.0	11.9
	32	57.0	39.1	33.9	29.4	31.9	17.5	16.8	15.4
260	22	26.8	18.4	15.9	13.8	14.4	7.93	7.61	6.97
	25	34.7	23.8	20.6	17.9	18.8	10.4	9.93	9.09
	28	43.5	29.8	25.8	22.4	24.0	13.2	12.7	11.6
	32	56.7	38.9	33.7	29.2	31.5	17.3	16.6	15.2

¹ Tabulated design values apply to ONE bolt in double shear

² To obtain adjusted lateral design values (**P'**, **Q'**), tabulated lateral design values (**P**, **Q**) for bolts shall be multiplied to all applicable adjustment factors based on National Design Specification for Wood Construction (NDS) 2015 Edition

³ When loaded at an angle θ to grain, lateral design value shall be computed using $N' = \frac{P'Q'}{P' \sin^2 \theta + Q' \cos^2 \theta}$

Table 619.1-2 (cont'd) Reference Values on One Bolt (Double Shear) in Seasoned Wood Loaded at Both Ends

Length of Bolt in Main Member L (mm)	Diameter of Bolt d (mm)	Loaded parallel to grain (0°)				Loaded perpendicular to grain (90°)			
		Design value, P , per bolt, kN				Design value, Q , per bolt, kN			
		Species Group				Species Group			
		I	II	III	IV	I	II	III	IV
280	25	34.6	23.8	20.6	17.9	18.1	9.94	9.53	8.73
	28	43.4	29.9	25.8	22.4	23.0	12.6	12.1	11.1
	32	56.9	39.0	33.8	29.3	30.8	17.0	16.3	14.9
290	25	34.7	23.8	20.6	17.9	17.8	9.79	9.39	8.60
	28	43.5	29.9	25.9	22.4	22.7	12.5	12.0	11.0
	32	56.8	39.0	33.7	29.3	30.4	16.7	16.1	14.7
305	25	34.7	23.8	20.6	17.8	17.3	9.51	9.13	8.36
	28	43.4	29.8	25.8	22.4	22.2	12.2	11.7	10.7
	32	56.8	39.0	33.8	29.3	29.8	16.4	15.8	14.4

¹ Tabulated design values apply to ONE bolt in double shear

² To obtain adjusted lateral design values (**P'**, **Q'**), tabulated lateral design values (**P**, **Q**) for bolts shall be multiplied to all applicable adjustment factors based on National Design Specification for Wood Construction (NDS) 2015 Edition

³ When loaded at an angle θ to grain, lateral design value shall be computed using $N' = \frac{P'Q'}{P' \sin^2 \theta + Q' \cos^2 \theta}$

Table 619.1-3 Split Ring Connector Unit Reference Design Values ^{1,2}

Split ring diameter (mm)	Bolt Diameter (mm)	Number of faces of member with connectors on same bolt	Net thickness of member (mm)	Loaded parallel to grain (0°)				Loaded perpendicular to grain (90°)			
				Design value, P , per connector unit and bolt, kN				Design value, Q , per connector unit and bolt, kN			
				Species Group				Species Group			
				I	II	III	IV	I	II	III	IV
65	12	1	25 minimum	11.7	10.1	8.45	7.30	8.45	7.21	6.01	5.16
			40 or thicker	14.1	12.1	10.2	8.72	10.1	8.63	7.21	6.19
		2	40 minimum	10.8	9.34	7.83	6.72	7.79	6.67	5.56	4.76
			50 or thicker	14.1	12.1	10.2	8.72	10.15	8.63	7.21	6.19
100	20	1	25 minimum	18.2	15.6	13.0	11.2	12.6	10.9	9.08	7.83
			40	26.8	23.0	19.0	16.5	18.6	16.0	13.3	11.5
			thicker than 40	27.3	23.4	19.5	16.9	19.0	16.3	13.6	11.7
		2	40 minimum	18.3	15.7	13.1	11.3	13.3	10.9	9.08	7.83
			50	22.0	18.9	15.7	13.6	15.3	13.2	10.9	9.43
			65	25.9	22.2	18.5	16.0	18.0	15.5	12.9	11.1
			75 or thicker	27.3	23.4	19.5	16.9	19.0	16.3	13.6	11.7

¹ Tabulated design values apply to ONE split ring and bolt in single shear.

² To obtain adjusted lateral design values (**P'**, **Q'**), tabulated lateral design values (**P**, **Q**) for split ring connector units shall be multiplied to all applicable adjustment factors based on National Design Specification for Wood Construction (NDS) 2015 Edition

³ When loaded at an angle θ to grain, lateral design value shall be computed using $N' = \frac{P'Q'}{P' \sin^2 \theta + Q' \cos^2 \theta}$

Table 619.1-4 Shear Plate Connector Unit Reference Design Values ^{1, 2, 3, 4, 5}

Shear plate diameter (mm)	Bolt Diameter (mm)	Number of faces of member with connectors on same bolt	Net thickness of member (mm)	Loaded parallel to grain (0°)				Loaded perpendicular to grain (90°)			
				Design value, P , per connector unit and bolt, kN				Design value, Q , per connector unit and bolt, kN			
				Species Group				Species Group			
				I	II	III	IV	I	II	III	IV
65	20	1	40 minimum	13.8*	11.9	9.88	8.94	9.66	8.28	6.90	5.92
		2	40 minimum	10.8	9.26	7.70	6.67	7.52	6.45	5.38	4.63
			50	14.2*	12.2	10.1	8.72	9.88	8.50	7.03	6.10
			65 or thicker	14.8*	12.7	10.6	9.17	10.3	8.86	7.34	6.41
100	20 or 22	1	40 minimum	19.4	16.7	13.9	12.0	13.5	11.7	9.66	8.28
			45 or thicker	22.6*	19.4	16.2	14.0	15.7	13.5	11.3	9.79
		2	45 minimum	15.1	12.9	10.8	9.30	10.5	8.99	7.48	6.27
			50	16.9	14.4	12.0	10.4	11.7	10.1	8.37	7.25
			65	19.2	16.4	13.7	11.8	13.3	11.3	9.52	8.23
			75	21.5*	18.4	15.3	13.3	14.9	12.8	10.7	9.17
			90 or thicker	22.4*	19.2	16.0	13.8	15.6	13.3	11.2	9.61

¹ Tabulated design values apply to ONE shear plate and bolt in single shear.

² To obtain adjusted lateral design values (**P'**, **Q'**), tabulated lateral design values (**P**, **Q**) for split ring connector units shall be multiplied to all applicable adjustment factors based on National Design Specification for Wood Construction (NDS) 2015 Edition

³ When loaded at an angle θ to grain, lateral design value shall be computed using $N' = \frac{P' Q'}{P' \sin^2 \theta + Q' \cos^2 \theta}$

⁴ Allowable design values for shear plate connector units shall not exceed the following:

- (a) 65 mm shear plate 12.9 kN
- (b) 100 mm shear plate with 20 mm bolt 19.6 kN
- (c) 100 mm shear plate with 22 mm bolt 26.7 kN

The design values in Footnote 3 shall be permitted to be increased in accordance with the American Institute of Steel Construction (AISC) Manual of Steel Construction, 9th edition, Section 15.2 "Wind and Seismic Stresses", except when design loads have already been reduced by load combination factors.

⁵ Loads followed by an asterisk (*) exceed those permitted by Footnote 3, but are needed for determination of design values for other angles of load to grain. Footnote 3 limitations apply in all cases.

619.3 Nails and Spikes

619.3.1 Nailing Schedule

The number and size of nails connecting wood members shall not be less than that set forth in Tables 619.3-1 and 619.3-2. Other connections shall be fastened to provide equivalent strength.

619.3.2 Safe Lateral Strength

A common wire nail driven perpendicular to grain of the wood, when used to fasten wood members together, shall not be subjected to a greater load causing shear and bending than the safe lateral strength of the wire nail or spike as set forth in Table 619.3-3.

A wire nail driven parallel to the grain of the wood shall not be subjected more than two thirds of the lateral load allowed when driven perpendicular to the grain. Toenails shall not be subjected more than five sixths of the lateral load allowed for nails driven perpendicular to the grain.

619.3.3 Safe Resistance to Withdrawal

A wire nail driven perpendicular to grain of wood shall not be subjected to a greater load, tending to cause withdrawal, than the safe resistance of the nail to withdrawal, as set forth in Table 619.3-3.

619.3.4 Spacing and Penetration

Common wire nails shall have penetration into the piece receiving the point as set forth in Table 619.3-3. Nails or spikes for which the wire gauges or lengths are not set forth in Table 619.3-3 shall have a required penetration of not less than 11 diameters, and allowable loads may be interpolated. Design values shall be increased when the penetration of nails into the member holding the point is larger than the required by this item.

For wood-to-wood joints, the spacing center to center of nails in the direction of stress shall not be less than one half of the required penetration. Edge or end distances in the direction of stress shall not be less one half of the required penetration. All spacing and edge and end distances shall be such as to avoid splitting of the wood.

Holes for nails, where necessary to prevent splitting, shall be bored of a diameter smaller than that of the nails.

Table 619.3-1 Nailing Schedule

Connection	Nailing ¹
1. Joist to sill or girder, toenail	3 - 65 mm
2. Bridging to joist, toenail each end	2 - 65 mm
3. 25 mm x 150 mm subfloor or less to each joist, face nail	2 - 65 mm
4. Wider than 25 mm x 150 mm subfloor to each joist, face nail	3 - 65 mm
5. 50 mm subfloor to joist or girder, blind and face nail	2 - 90 mm
6. Sole plate to joist or blocking, typical face nail	90 mm at 400 mm o.c.
Sole plate to joist or blocking, at braced wall panels	3-90 mm per 400 mm
7. Top plate to stud, end nail	2-90 mm
8. Stud to sole plate	4-65 mm, toenail or 2-90 mm, end nail
9. Double studs, face nail	90 mm at 600 mm o.c.
10. Doubled top plates, typical face nail	90 mm at 400 mm o.c.
Double top plates, lap splice	8-90 mm
11. Blocking between joists or rafters to top plate, toenail	3-65 mm
12. Rim joist to top plate, toenail	65 mm at 150 mm o.c.
13. Top plates, laps and intersections, face nail	2-90 mm
14. Continuous header, two pieces	90 mm at 400 mm o.c. along each edge
15. Ceiling joists to plate, toenail	3-65 mm
16. Continuous header to stud, toenail	4-65 mm
17. Ceiling joists, laps over partitions, face nail	3-90 mm
18. Ceiling joists to parallel rafters, face nail	3-90 mm
19. Rafter to plate, toenail	3-65 mm
20. 25 mm brace to each stud and plate, face nail	2-65 mm
21. 25 mm x 200 mm sheathing or less to each bearing, face nail	2-65 mm
22. Wider than 25 mm x 200 mm sheathing to each bearing, face nail	3-65 mm
23. Built-up corner studs	90 mm at 600 mm o.c.
24. Built-up girder and beams	100 mm at 800 mm o.c. at top and bottom and staggered 2-100 mm at ends and at each splice 2-90 mm at each bearing
25. 50 mm planks	
26. Wood structural panels and particleboard ² :	
Subfloor and wall sheathing (to framing):	
12 mm and less	50 mm ³
16 mm – 20 mm	65 mm ⁴ or 50 mm ⁵
22 mm – 25 mm	65 mm ³
28 mm – 32 mm	75 mm ⁴ or 65 mm ⁵
Combination subfloor-underlayment (to framing):	
20 mm and less	50 mm ⁵
22 mm – 25 mm	65 mm ⁵
30 mm – 32 mm	75 mm ⁴ or 65 mm ⁵
27. Panel siding (to framing) ² :	
12 mm or less	50 mm ⁶
16 mm	65 mm ⁶
28. Fiberboard sheathing ⁷ :	
12 mm	10 mm x 40 mm ⁸ 50 mm ⁴
	10 mm x 30 mm ⁹
	10 mm x 45 mm ⁸
	65 mm ⁴
	10 mm x 40 mm ⁹
29. Interior paneling	
6 mm	40 mm ¹⁰
10 mm	50 mm ¹¹

Notes for Table 619.3-1:

¹ Common or box nails may be used except where otherwise stated.² Nails spaced at 150 mm on center at edges, 300 mm at intermediate supports except 150 mm at all supports where spans are 1200 mm or more.

For nailing of wood structural panel and particleboard diaphragms and shear walls, refer to Sections 614.3.3 and 614.4. Nails for wall sheathing may be common, box or casing.

³ Common or deformed shank.

⁴ Common.⁵ Deformed shank.⁶ Corrosion-resistant siding or casing nails conforming to the requirements of Section 603.3.⁷ Fasteners spaced 75 mm on center at exterior edges and 150 mm on center at intermediate supports.⁸ Corrosion-resistant roofing nails with 10 mm head and 40 mm length for 12 mm sheathing and 45 mm length for 20 mm sheathing conforming to the requirements of Section 603.3.⁹ Corrosion-resistant staples with nominal 10 mm crown 30 mm length for 12 mm sheathing and 40 mm length for 20 mm sheathing conforming to the requirements of Section 603.3.¹⁰ Panel supports at 400 mm (500 mm if strength axis in the long direction of the panel, unless otherwise marked). Casing or finish nails spaced 150 mm on panel edges, 300 mm at intermediate supports.¹¹ Panel supports at 600 mm. Casing or finish nails spaced 150 mm on panel edges, 300 mm at intermediate supports.Table 619.3-2 Wood Structural Panel Roof Sheathing Nailing Schedule ¹

WIND REGION	NAILS	PANEL LOCATION	ROOF FASTENING ZONE ²		
			1	2	3
			Fastening Schedule (mm on center)		
Greater than 145 kph	65 mm common	Panel edges ³	150	150	100 ⁴
		Panel Field	150	150	150 ⁴
Greater than 129 kph to 145 kph	65 mm common	Panel edges ³	150	150	100
		Panel Field	300	150	150
129 kph or less	65 mm common	Panel edges ³	150	150	150
		Panel Field	300	300	300

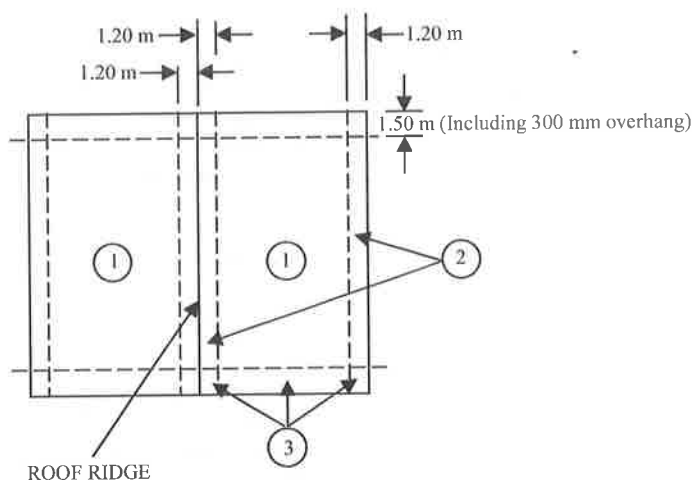
¹ Applies only to mean roof heights up to 10.5 m. For mean roof heights over 10.5 m., the nailing shall be designed² The roof fastening zones are shown in Figure 619.3-1.³ Edge spacing also applies over roof framing at gable-end walls.⁴ Use 65mm ring-shank nails in this zone if mean roof height is greater than 7.50 m

Figure 619.3-1 Roof Fastening Zones

Table 619.3-3 Allowable Loads in Seasoned Wood for Common Wire Nails and Spikes (Normal Duration)

Size of Nail or Spike (mm)				Withdrawal Load from Side Grain per 25 mm of Penetration of Nail or Spike into the Member Holding the Point (N)				Lateral Load in Side Grain, (N)			
Designation	Length (mm)	Diameter (mm)		Species Group				Species Group			
				I	II	III	IV	I	II	III	IV
N	150	150	6.5	805	550	340	215	1320	1135	930	775
	125	140	6.0	750	510	315	200	1180	1010	830	695
A	110	125	5.75	690	470	290	180	1045	895	735	615
	105	115	5.25	685	435	265	170	920	790	650	540
I	100	100	4.75	590	400	245	155	825	705	580	485
	90	90	4.00	495	340	210	130	640	550	450	375
L	75	75	3.75	455	310	190	120	555	480	395	330
	65	65	3.25	400	275	170	105	465	400	325	275
S	50	50	3.00	345	235	145	90	370	320	260	220
S	3/8	215	9.5	1035	705	435	275	2020	1735	1425	1190
	3/8	180	8.0	860	590	360	230	1535	1320	1085	905
P	150	150	7.0	780	535	325	205	1325	1140	935	780
	140	140	7.0	780	536	325	205	1325	1140	935	780
I	125	125	6.75	725	495	305	195	1185	1020	840	700
	110	115	6.0	675	460	280	180	1060	910	750	625
K	100	100	5.75	620	425	260	165	940	805	665	555
	90	90	5.25	570	390	240	150	830	710	585	490
E	80	80	5.0	530	360	220	140	740	635	525	435
S	75	75	5.0	530	360	220	140	740	635	525	435

619.4 Joist Hangers and Framing Anchors

Connections depending upon joist hangers or framing anchors, ties and other mechanical fastenings not otherwise covered may be used where approved.

619.5 Miscellaneous Fasteners

619.5.1 Drift Bolts or Drift Pins

Connections involving the use of drift bolts or pins shall be designed in accordance with the provisions set forth in this Chapter.

619.5.1.1 Withdrawal Design Values

Drift bolt and drift pin connections loaded in withdrawal shall be designed in accordance with good engineering practice. Figures 619.5-1 to 619.5-3 are examples of withdrawal connections.

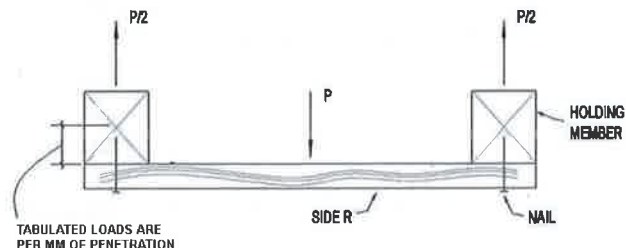


Figure 619.5-1 Basic Withdrawal Connection

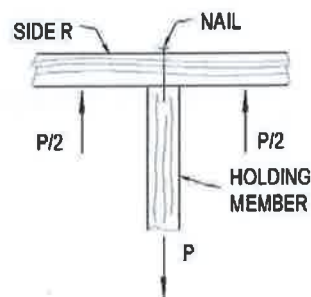


Figure 619.5-2 Withdrawal from End Grain (not allowed)

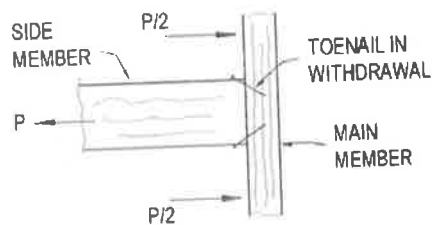


Figure 619.5-3 Toenail Connection Withdrawal from Side Grain

619.5.1.2 Lateral Design Values

Allowable lateral design values for drift bolts and drift pins driven in the side grain of wood shall not exceed 75 percent of the allowable lateral design values for common bolts of the same diameter and length in main member.

Additional penetration of pin into members should be provided in lieu of the washer, head and nut on a common bolt.

619.5.2 Spike Grids

Wood-to-wood connections involving spike grids for lateral load transfer shall be designed in accordance with good engineering practice.

619.5.3 Wood Screws and Lag Screws

Wood and lag screws shall be used where there is limited penetration, especially in a withdrawal design, as these provide greater resistance. Design of the screws shall be in accordance with Table 619.5-1 and the provisions set forth in this Chapter.

Table 619.5-1 Allowable Loads in Wood Screws for Seasoned Wood (Normal Duration)

Screw Size	Withdrawal Load from Side Grain per 25 mm of Penetration of Threaded Portion, (N)				Lateral Load in Side Grain, (N)			
Diameter (mm)	Species Group				Species Group			
	I	II	III	IV	I	II	III	IV
9.5	2695	1985	1370	950	3100	2665	2190	1825
8.0	2315	1710	1180	820	2295	1970	1620	1350
7.5	2130	1570	1085	750	1935	1665	1370	1140
7.0	1940	1430	985	685	1610	1380	1135	945
6.0	1750	1290	890	620	1315	1130	925	770
5.5	1565	1155	795	550	1045	900	740	615
5.0	1375	1015	700	485	810	695	570	475
4.5	1280	945	650	450	700	605	495	415
4.0	1185	875	605	420	605	520	425	355
3.8	1095	805	555	385	510	440	360	300
3.5	1000	735	510	355	425	365	300	250

SECTION 620

CONVENTIONAL LIGHT-FRAME CONSTRUCTION DESIGN PROVISIONS

620.1 General

The requirements in this section are intended for conventional light-frame construction. Other methods may be used provided a satisfactory design is submitted showing compliance with other provisions of this code.

Only the following occupancies may be constructed in accordance with this division:

1. One-, two- or three-story residential buildings.
2. One-story Occupancy Category IV buildings, as defined in Table 103-1, when constructed on a slab-on-grade floor.
3. Category V Occupancies
4. Top-story walls and roofs of Occupancy Category IV buildings not exceeding two stories of wood framing.
5. Interior non-load bearing partitions, ceilings and curtain walls in all occupancies.

Other approved repetitive wood members may be used in lieu of solid-sawn lumber in conventional construction provided these members comply with the provisions of this code.

620.2 Design of Portions

620.3 Additional Requirements for Conventional Construction in High-wind Areas

Provisions for conventional construction in high-wind areas shall apply when specifically adopted.

620.4 Additional Requirements for Conventional Construction in Seismic Zone 2

620.4.1 Braced Wall Lines

In areas under Seismic Zone 2 and where the basic wind speed obtained from Chapter 2 is not greater than 125 kph, buildings shall be provided with exterior and interior braced wall lines not exceeding 10.0 m on center in both the longitudinal and transverse directions in each story.

620.4.2 Veneer

Anchored masonry and stone wall veneer shall not exceed 125 mm in thickness.

620.4.3 Unusually Shaped Buildings

When of unusual shape, buildings of light-frame construction shall have a lateral-force-resisting system designed to resist the forces specified in Chapter 2. One or more of the following shall be considered to constitute an unusual shape:

620.4.3.1 When exterior braced wall panels, as required by Section 620.9.3, are not in one plane vertically from the foundation to the uppermost story in which they are required.

Exceptions:

Floors with cantilevers or setbacks not exceeding four times the nominal depth of the floor joists may support braced wall panels provided:

1. Floor joists are 50 mm by 250 mm or larger and spaced at not more than 400 mm on center.
2. The ratio of the adjacent span to the cantilever is at least 2 to 1.
3. Floor joists at ends of braced wall panels are doubled.
4. A continuous rim joists is connected to ends of all cantilevered joists. The rim joist may be spliced using a metal tie not less than 1.5 mm and 38 mm wide fastened with six 90 mm nails.
5. Gravity loads carried at the end of cantilevered joists are limited to uniform wall and roof load and the reactions from headers having a span of 2.40 m or less.

620.4.3.2 When a section of floor or roof is not laterally supported by braced wall lines on all edges.

Exception:

Portions of roofs or floors which do not support braced wall panels above may extend up to 1.80 m beyond a braced wall line.

620.4.3.3 When the end of a required braced wall panel extends more than 300 mm over an opening in the wall below. This provision is applicable to braced wall panels offset in plane and to braced wall panels offset out of plane as permitted by Section 620.5.3.1 with this exception.

Exception:

Braced wall panels may extend over an opening not more than 2.40 m. in width when the header is a 100 mm by 300 mm or larger member.

620.4.3.4 When an opening in a floor or roof exceeds the lesser of 3.60 m or 50 percent of the least floor or roof dimension.

620.4.3.5 Construction where portions of a floor level are vertically offset such that the framing members on either side of the offset cannot be lapped or tied together in an approved manner as required by Section 620.7.3.

Exception:

Framing supported directly by foundations.

620.4.3.6 When braced wall lines do not occur in two perpendicular directions.

620.4.3.7 Other configurations which, in the opinion of the building official, create irregularities or discontinuities which are not addressed by this Section.

620.4.4 Lumber Roof Decks

Lumber roof decks shall have solid sheathing.

620.4.5 Interior Braced Wall Support

In one-story buildings, interior braced wall lines shall be supported on continuous foundations at intervals not exceeding 15.0 m. In buildings more than one-story in height, all interior braced wall panels shall be supported on continuous foundations.

Exception:

Two-story buildings may have interior braced wall lines supported on continuous foundations at intervals not exceeding 15.0 m. provided:

1. *Cripple wall height does not exceed 1.20 m.*
2. *First-floor braced wall panels are supported on doubled floor joists, continuous blocking, or floor beams.*
3. *Distance between bracing lines does not exceed twice the building width parallel to the braced wall line.*

620.5 Girders

Unless otherwise permitted by provisions in NSCP Volume III - Housing, girders for single-story construction or girders supporting loads from a single floor shall not be less than 100 mm by 150 mm for spans 1.80 m or less, provided that girders are spaced not more than 2.40 m on center. Other girders shall be designed to support the loads specified in this code. Girder end joints shall occur over supports. When a girder is spliced over a support, an adequate tie shall be provided. The end of beams or girders supported on masonry or concrete shall not have less than 75 mm of bearing.

620.6 Floor Joists**620.6.1 General**

The limits of defects by grade in joists and planks for seasoned wood are set forth in Table 615.1-1.

620.6.2 Bearing

Except where supported on a 25 mm by 100 mm ribbon strip and nailed to the adjoining stud, the ends of each joist shall not have less than 38 mm of bearing on wood or metal, or less than 75 mm on masonry.

620.6.3 Framing Details

Joists shall be supported laterally at the ends and at each support by solid blocking except where the ends of joists are nailed to a header, band or rim joist or to an adjoining stud or by other approved means. Solid blocking shall not be less 50 mm in thickness and the full depth of joist.

Notches on the ends of joists shall not exceed one-fourth the joist depth. Holes bored in joists shall not be within 50 mm of the top or bottom of the joist and the diameter of any such hole shall not exceed one-third the depth of the joist. Notches in the top or bottom of joists shall not exceed one-sixth the depth and shall not be located in the middle third of the span.

Joist framing from opposite sides of a beam, girder or partition shall be lapped at least 75 mm or the opposing joists shall be tied together in an approved manner.

Joists framing into the side of a wood girder shall be supported by framing anchors or on ledger strips not less than 50 mm by 50 mm.

620.6.4 Framing Around Openings

Trimmer and header joists shall be doubled, or of lumber of equivalent cross section, when the span of the header exceeds 1.20 m. The ends of header joists more than 1.80 m long shall be supported by framing anchors or joist hangers unless bearing on a beam, partition or wall. Tail joists over 3.60 m long shall be supported at header by framing anchors or on ledger strips not less than 50 mm by 50 mm.

620.6.5 Supporting Bearing Partitions

Bearing partitions perpendicular to joists shall not be offset from supporting girders, walls or partitions more than the joist depth.

Joists under and parallel to bearing partitions shall be doubled.

620.6.6 Blocking

Floor joists shall be blocked when required by the provisions of Section 617.4.1.2.

620.7 Subflooring

620.7.1 Lumber Subfloor

Sheathing used as a structural sub-floor shall conform to the limitations set forth in Table 611.1-1.

Joints in subflooring shall occur over supports unless end-matched lumber is used in which case each piece shall bear on at least two joists.

Subflooring may be omitted when joist spacing does not exceed 400 mm and 25 mm nominal tongue-and-groove wood strip flooring is applied perpendicular to the joists.

620.7.2 Wood Structural Panels

Where used as structural subflooring, wood structural panels shall be as set forth in Tables 611.1-2 and 611.1-3. Wood structural panel combination subfloor underlayment shall have maximum spans as set forth in Table 611.1-4.

When wood structural panel floors are glued to joists with an adhesive in accordance with the adhesive manufacturer's directions, fasteners may be spaced a maximum of 300 mm on center at all supports.

620.7.3 Plank Flooring

Plank flooring shall be designed in accordance with the general provisions of this code.

In lieu of such design, 50 mm tongue-and-groove planking may be used in accordance with Table 620.9-1. Joints in such planking may be randomly spaced, provided the system is applied to not less than three continuous spans, planks are center-matched and end-matched or splined, each plank bears on at least one support and joints are separated by at least 600 mm in adjacent pieces. 25 mm nominal strip square-edged flooring; 13 mm tongue-and-groove flooring or 9 mm wood structural panel shall be applied at right angles to the span of the planks. The 9 mm plywood shall be applied with the face grain at right angles to the span of the planks.

620.7.4 Particleboard

Where used as structural subflooring or as combined subfloor underlayment, particleboard shall be as set forth in Table 611.1-5.

620.8 Particleboard Underlayment

In accordance with approved recognized standards, particleboard floor underlayment shall conform to Type PBU. Underlayment shall not be less than 6 mm in thickness and shall be identified by the grade mark of an approved inspection agency. Underlayment shall be installed in accordance with this code and as recommended by the manufacturer.

620.9 Wall Framing

620.9.1 Size, Height and Spacing

The size, height and spacing of studs shall be in accordance with Table 620.9-2 except that utility grade studs shall not be spaced more than 400 mm on center, or support more than a roof and ceiling, or exceed 2.40 m in height for exterior walls and load-bearing walls or 3.00 m for interior non load-bearing walls.

Table 620.9-1 Allowable Spans for 50 mm Tongue and Groove Decking

Table 620.9-1 Allowable Spans for 50 mm Tongue and Groove Decking

SPAN ¹ (mm)	LIVE LOAD (kPa)	<i>f</i> (MPa)	Deflection Limit	
			1/240	1/360
			<i>E</i> x 10 ³ MPa	
ROOF				
1200	0.96	1.10	1.17	1.76
	1.44	1.45	1.76	2.65
	1.92	1.86	2.30	3.52
1350	0.96	1.38	1.67	2.10
	1.44	1.86	2.50	2.79
	1.92	2.41	3.33	5.00
1500	0.96	1.72	2.29	3.45
	1.44	2.27	3.41	5.11
	1.92	2.89	4.55	6.89
1650	0.96	2.07	3.05	4.55
	1.44	2.76	4.56	6.88
	1.92	3.44	6.09	9.16
1800	0.96	2.48	3.96	5.94
	1.44	3.31	5.94	8.92
	1.92	4.14	7.92	11.9
1950	0.96	2.89	4.10	6.15
	1.44	3.86	6.15	9.23
	1.92	4.82	8.20	11.9
2100	0.96	3.38	6.27	9.37
	1.44	4.48	9.44	13.8
	1.92	5.58	12.5	18.8
2250	0.96	3.86	7.75	11.6
	1.44	5.17	11.6	17.4
	1.92	6.41	15.5	23.3
2400	0.96	4.41	9.37	14.06
	1.44	5.86	14.1	21.1
FLOOR				
1200	1.92	5.79		6.90
1350	1.92	6.55		9.00
1500	1.92	7.30		11.0

¹ Spans are based on simple beam action with 0.50 kN/m² dead load and provisions for a 1300 N concentrated load on a 300 mm width of floor decking. Random lay-up permitted in accordance with the provisions of Section 620.7.3 or 620.9.11.9. Lumber thickness assumed at 40 mm, net.

Table 620.9-2 Size, Height and Spacing of Wood Studs

Stud Size	Bearing Walls				Non-Bearing Walls	
	Laterally Unsupported Stud Height ¹	Supporting Roof and Ceiling Only	Supporting One Floor, Roof and Ceiling	Supporting Two Floors, Roof and Ceiling	Laterally Unsupported Stud Height ¹	Spacing
(mm)	(mm)	Spacing (mm)			(m)	(mm)
50 x 75 ²	-	-	-	-	3.0	400
50 x 100	250	600	400	-	4.2	600
75 x 100	250	600	600	400	4.2	600
50 x 125	250	600	600	-	4.8	600
50 x 150	250	600	600	400	6.0	600

¹ Listed heights are distances between points of lateral support placed perpendicular to the plane of the wall. Increases in unsupported height are permitted where justified by an analysis.

² Shall not be used in exterior walls.

620.9.2 Framing Details

Studs shall be placed with their wide dimension perpendicular to the wall. Not less than three studs shall be installed at each corner of an exterior wall.

Exceptions:

At corners, a third stud may be omitted through the use of wood spacers or backup cleats of 9 mm wood structural panel, 9 mm Type M "Exterior Glue" particle-board, 25 mm lumber or other approved devices that will serve as an adequate backing for the attachment of facing materials. Where fire resistance ratings or shear values are involved, wood spacers, backup cleats or other devices shall not be used unless specifically approved for such use.

Bearing and exterior wall studs shall be capped with double top plates installed to provide overlapping at corners and at intersections with other partitions. End joints in double top plates shall be offset at least 2.40 m.

Exceptions:

A single top plate may be used, provided the plate is adequately tied at joints, corners and intersecting walls by at least the equivalent of 75 mm by 150 mm by 0.9 mm galvanized steel that is nailed to each wall or segment of wall by six 65 mm nails or equivalent, provided the rafters, joists or trusses are centered over the studs with a tolerance of no more than 25 mm.

When bearing studs are spaced at 600 mm intervals and top plates are less than 50 mm by 150 mm or 70 mm by 100 mm members and when the floor joists, floor trusses or roof trusses which they support are spaced at more than 400 mm intervals, such joists or trusses shall bear within 125 mm of the studs beneath or a third plate shall be installed.

Interior non-bearing partitions may be capped with a single top plate installed to provide overlapping at corners and at intersections with other walls and partitions. The plate shall be continuously tied at joints by solid blocking at least 400 mm in length and equal in size to the plate or by 3 mm by 38 mm metal ties with spliced sections fastened with two 90 mm nails on each side of the joint

Studs shall have full bearing on a plate or sill not less than 50 mm thick having a width not less than that of the wall studs.

620.9.3 Bracing

Braced wall lines shall consist of braced wall panels which meet the requirements for location, type and amount of bracing specified in Table 6.24 and are in line or offset from each other by not more than 1.20 m. Braced wall panels shall start at not more than 2.40 m from each end of a braced wall line. All braced wall panels shall be clearly indicated on the plans. Construction of braced wall panels shall be by one of the following methods:

1. Nominal 25 mm by 100 mm continuous diagonal braces let into top and bottom plates and intervening studs, placed at an angle not more than 60 degrees or less than 45 degrees from the horizontal, and attached to the framing in conformance with Table 619.3-1.
2. Wood boards of 16 mm net minimum thickness applied diagonally on studs spaced not over 600 mm on center.
3. Wood structural panel sheathing with a thickness not less than 8 mm for 400 mm stud spacing and not less than 9 mm for 600 mm stud spacing in accordance with Tables 609.3-1 and 620.9-3.
4. Fiberboard sheathing 1.20 m by 2.40 m panels not less than 13 mm thick applied vertically on studs spaced not over 406 mm on center when installed in accordance with Section 614.6 and Table 614.6-1.
5. Gypsum board (sheathing 13 mm thick by 1.20 m wide, wallboard or veneer base) on studs spaced not over 600 mm on center and nailed at 175 mm on center with nails as required by Table 620.9-5.
6. Particleboard wall sheathing panels where installed in accordance with Table 620.9-6.
7. Portland cement plaster on studs spaced 400 mm on center installed in accordance with Table 620.9-5.
8. Hardboard panel siding when installed in accordance with Section 609.6 and Table 609.6-1.

User Note: Method 1 is not permitted in the Philippines.

For cripple wall bracing, see Section 620.9.5. For Methods 2, 3, 4, 6 and 8, each braced panel must be at least 1.20 m in length, covering three stud spaces where studs are spaced 400 mm apart and covering two stud spaces where studs are spaced 600 mm apart.

For Method 5, each braced wall panel must be at least 2.40 m in length when applied to one face of a braced wall panel and 1.20 m when applied to both faces.

All vertical joints of panel sheathing shall occur over studs. Horizontal joints shall occur over blocking equal in size to the studding except where waived by the installation requirements for the specific sheathing materials.

Braced wall panel sole plates shall be nailed to the floor framing and top plates shall be connected to the framing above in accordance with Table 619.3-1. Sills shall be bolted to the foundation or slab. Where joists are perpendicular to braced wall lines above, blocking shall be provided under and in line with the braced wall panels.

620.9.4 Alternate Braced Panels

Any braced wall panel required by Section 620.9.3 may be replaced by an alternate braced wall panel constructed in accordance with the following:

1. In one-story buildings, each panel shall have a length of not less than 800 mm and a height of not more than 3.0 m. Each panel shall be sheathed on one face with 9 mm plywood sheathing nailed with 65mm common or galvanized box nails in accordance with Table 619.3-1 and blocked at all plywood edges. Two anchor bolts installed shall be provided in each panel. Anchor bolts shall be placed at panel quarter points. Each panel end stud shall have a tie-down device fastened to the foundation, capable of providing an approved uplift capacity of not less than 820 kN. The tie-down device shall be installed in accordance with the manufacturer's recommendations. The panels shall be supported directly on a foundation or on floor framing supported directly on a foundation which is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than one 12 mm bar top and bottom.
2. In the first story of two-story buildings, each braced wall panel shall be in accordance with Section 620.9.4, Item 1, except that the plywood sheathing shall be provided on both faces, three anchor bolts shall be placed at one-fifth points, and tie-down device uplift capacity shall not be less than 13.5 kN.

Table 620.9-2 Braced Wall Panels¹

Seismic Zone	Condition	Construction Method ^{2,3}								Braced Panel Location and Length ⁴
		1	2	3	4	5	6	7	8	
2	One-story, top of two or three-story	X	X	X	X	X	X	X	X	Each end and not more than 7.5 m on center
	First-story of two-story or second-story of three-story	X	X	X	X	X	X	X	X	
	First-story of three-story		X	X	X	X ⁵	X	X	X	
4	One-story, top of two-story or three-story		X	X	X	X	X	X ⁶	X	Each end and not more than 7.5 m on center
	First-story of two-story or second of three-story		X	X	X	X ⁵	X	X ⁶	X	Each end and not more than 7.5 m on center but not less than 25% of building length ⁷
	First-story of three-story		X	X	X	X ⁵	X	X ⁶	X	Each end and not more than 7.5 m on center but not less than 40% of building length ⁷

¹ This table specifies minimum requirements for braced panels which form interior or exterior braced wall lines.² See Section 620.9.3 for full description.³ See Section 620.9.4 for alternate braced panel requirement.⁴ Building length is the dimension parallel to the braced wall length.⁵ Gypsum wallboard applied to supports at 400 mm on center.⁶ Not permitted for bracing cripple walls in Seismic Zone 4. See Section 620.9.5.⁷ The required lengths shall be doubled for gypsum board applied to only one face of a braced wall panel.

Table 620.9-3 Cripple Wall Bracing

Seismic Zone	Condition	Amount of Cripple Wall Bracing ^{1,2}	
		(mm)	
2	One-story above cripple wall	10mm wood structural panel with 65mm at 150 / 300 mm nailing on 30 percent of wall length minimum	
	Two-story above cripple wall	10mm wood structural panel with 65mm at 100 / 300 mm nailing on 40 percent of wall length minimum or 10mm wood structural panel with 65mm at 150 / 300 mm nailing on 60 percent of wall length minimum	
4	One-story above cripple wall	10mm wood structural panel with 65mm at 150 / 300 mm nailing on 60 percent of wall length minimum	
	Two-story above cripple wall	10mm wood structural panel with 65mm at 100 / 300 mm nailing on 50 percent of wall length minimum or 10mm wood structural panel with 65mm at 150 / 300 mm nailing on 75 percent of wall length minimum	

¹ Braced panel length shall be at least two times the height of the cripple wall, but not less than 1200 mm.² All panels along a wall shall be nearly equal in length and shall be nearly equally spaced along the length of the wall.Table 620.9-4 Wood Structural Panel Wall Sheathing¹
(Not exposed to the weather, strength axis parallel or perpendicular to studs)

Minimum Thickness (mm)	Panel Span Rating	Stud Spacing (mm)		
		Siding Nailed to Studs	Sheathing under Coverings Specified in Section 620.9.3	
			Sheathing Parallel to Studs	Sheathing Perpendicular to Studs
10	16/0, 16/0, 20/0 Wall – 400 mm o.c.	400	-	400
10, 12	16/0, 20/0, 24/0, 32/16 Wall – 600 mm o.c.	600	400	600
10, 12	24/0, 24/16, 32/16 Wall – 600 mm. o. c.	600	600	600

¹ In reference to Section 620.9.3, blocking of horizontal joints is not required.

Table 620.9-5 Allowable Shear for Wind or Seismic Forces in Newtons per m for Vertical Diaphragms of Lath and Plaster or Gypsum Board Frame Wall Assemblies¹

Type of Material	Thickness of Material (mm)	Wall Construction	Nail Spacing ² Maximum (mm)	Shear Value	Minimum Nail Size ³ (mm)	
1. Expanded metal, or woven wire lath and portland cement plaster	22 mm	Unblocked	150	2628	40 mm long, 10 mm head Staple, 22 mm legs	
2. Gypsum lath	10 mm lath and 12 mm plaster	Unblocked	125	1460	Staple, 30 mm long, 6 mm head, plasterboard blued nail	
3. Gypsum sheathing board	12 mm x 600 mm x 2400 mm	Unblocked	100	1095	45 mm long, 10 mm head, diamond-point, galvanized	
	12 mm x 1200 mm	Blocked	100	2555		
	12 mm x 1200 mm	Unblocked	175	1460		
4. Gypsum wallboard or veneer base	12 mm	Unblocked	175	1460	2 mm dia., 40 mm long, 6 mm head) or wallboard (2 mm dia. 40 mm long, 6 mm head)	
			100	1825		
	16 mm	Blocked	175	1825	(2.5 mm dia., 45 mm long, 6 mm head) or wallboard (2.5 mm dia. 50 mm long, 6 mm head)	
			100	2190		
		Unblocked	175	1679		
			100	2117		
		Blocked	175	2117		
			100	2555		
		Blocked Two ply	Base ply: 225 Face ply: 175	3650		
				Base ply – (2.5 mm dia. 50 mm long, 6 mm head) or wallboard (2.3 mm dia. 50 mm long, 6 mm head) Face ply – (3.0 mm dia., 60 mm long, 6 mm head) or wallboard (3.0 mm dia., 60 mm long, 10 mm head)		

¹ These vertical diaphragms shall not be used to resist loads imposed by masonry or concrete construction. Values shown are for short-term loading due to wind or due to seismic loading. Values shown must be reduced 25 percent for normal loading. The values shown in Items 2, 3 and 4 shall be reduced 50 percent for loading due to earthquake in Seismic Zone 4.

² Applies to nailing at all studs, top and bottom plates, and blocking.

³ Alternate nails may be used if their dimensions are not less than the specified dimension.

Table 620.9-6 Allowable Spans for Particleboard Wall Sheathing 1
(Not exposed to the weather, long dimension of the panel parallel or perpendicular to studs)

GRADE	THICKNESS (mm)	Stud Spacing (mm)	
		Siding Nailed to Studs	Sheathing under Coverings Specified in Section 620.9.3 Parallel or Perpendicular to Studs
M-1	10	400	400
M-S			
M-2 "Exterior Glue"	12	400	400

¹ In reference to Section 620.9.3, blocking of horizontal joints is not required.

620.9.5 Cripple Walls

Foundation cripple walls shall be framed of studs not less in size than the studding above with a minimum length of 350 mm, or shall be framed of solid blocking. When exceeding 1.20 m in height, such walls shall be framed of studs having the size required for an additional story.

Cripple walls having a stud height exceeding 350 mm shall be braced in accordance with Table 620.9-4. Solid blocking or wood structural panel sheathing may be used to brace cripple walls having a stud height of 350 mm or less. In Seismic Zone 4, Method 7 is not permitted for bracing any cripple wall studs.

Spacing of boundary nailing for required wall bracing shall not exceed 150 mm on center along the foundation plate and the top plate of the cripple wall. Nail size, nail spacing for field nailing and more restrictive boundary nailing requirements shall be as required elsewhere in the Chapter for the specific bracing material used.

620.9.6 Headers

Headers and lintels shall conform to the requirements set forth in this paragraph and together with their supporting systems shall be designed to support the loads specified in this code. All openings 1.20 m wide or less in bearing walls shall be provided with headers consisting of either two pieces of 50 mm framing lumber placed on edge and securely fastened together or 100 mm lumber of equivalent cross section. All openings more than 1.20 m wide shall be provided with headers or lintels. Each end of lintel or header shall have a length of bearing of not less than 38 mm for the full width of the lintel.

620.9.7 Pipes in Walls

Stud partitions containing plumbing, heating, or other pipes shall be so framed and the joists underneath so spaced as to give proper clearance for the piping. Where a partition containing such piping runs parallel to the floor joists, the joists underneath such partitions shall be doubled and spaced to permit the passage of such pipes and shall be bridged. Where plumbing, heating or other pipes are placed in or partly in a partition, necessitating the cutting of the soles or plates, a metal tie not less than 1.5 mm (16 galvanized gage) and 38 mm wide shall be fastened to each plate across and to each side of the opening with not less than six 90 mm nails.

620.9.8 Bridging

Unless covered by interior or exterior wall coverings or sheathing meeting the minimum requirements of this Chapter, all stud partitions or walls with studs having a height-to-least thickness ratio exceeding 50 shall have bridging not less than 50 mm in thickness and of the same width as the studs fitted snugly and nailed thereto to provide adequate lateral support.

620.9.9 Cutting and Notching

In exterior walls and bearing partitions, any wood stud may be cut or notched to a depth not exceeding 25 percent of its width. Cutting or notching of studs to a depth not greater than 40 percent of the width of the stud is permitted in nonbearing partitions supporting no loads other than the weight of the partition.

620.9.10 Bored Holes

Bored holes may be permitted in any wood stud provided the holes are not greater than 40 percent of the stud width. Bored holes not greater than 60 percent of the width of the study is permitted in nonbearing partitions or in any wall where each bored stud is doubled, provided not more than two such successive doubled studs are so bored.

In no case shall the edge of the bored hole be nearer than 16 mm to the edge of the stud. Bored holes shall not be located at the same section of stud as a cut or notch.

620.9.11 Roof and Ceiling Framing

620.9.11.1 General

The framing details required in this section apply to roofs having a minimum slope of 3 units vertical in 12 units horizontal (25% slope) or greater. When the roof slope is less than 3 units vertical in 12 units horizontal (25% slope), members supporting rafters and ceiling joists such as ridge board, hips and valleys shall be designed as beams.

620.9.11.2 Framing

Rafters shall be framed directly opposite each other at the ridge. There shall be a ridge board at least 25 mm nominal thickness at all ridges and not less in depth than the cut end of the rafter. At all valleys and hips there shall be a single valley or hip rafter not less than 50 mm nominal thickness and not less than the cut of the rafter.

620.9.11.3 Notches and Holes

Notching at the ends of rafters or ceiling joists shall not exceed one fourth the depth. Notches in the top or bottom of the rafter or ceiling joist shall not exceed one sixth the depth and shall not be located in the middle one third of the span, except that a notch not exceeding one third of the depth is permitted in the top of the rafter or ceiling joist not further from the face of the support than the depth of the member.

Holes bored in rafters or ceiling joists shall not be within 50 mm of the top and bottom and their diameter shall not exceed one third the depth of the member.

620.9.11.4 Framing Around Openings

Trimmer and header rafters shall be doubled, or of lumber of equivalent cross section, when the span of the header exceeds 1.20 m. The ends of header rafters more than 1.80 m long shall be supported by framing anchors or rafter hangers unless bearing on a beam, partition or wall.

620.9.11.5 Rafter Ties

Rafter shall be nailed to adjacent ceiling joists to form a continuous tie between exterior walls when such joists are parallel to the rafters. Where not parallel, rafter shall be tied to 25 mm by 100 mm (nominal) minimum-size crossties. Rafter ties shall be spaced not more than 1.20 m on center.

620.9.11.6 Purlins

The maximum span of 50mm by 150 mm purlins shall be 1.80 m but in no case shall the purlins be smaller than 50 mm by 100 mm members. The unbraced length of struts shall not exceed 2.40 m and the minimum slope of the struts shall be 45 degrees from the horizontal.

620.9.11.7 Blocking

Roof rafters and ceiling joists shall be supported laterally to prevent rotation and lateral displacement when required by Section 616. Roof trusses shall be supported laterally at points of bearing by solid blocking or by other equivalent means to prevent rotation and lateral displacement.

620.9.11.8 Roof Sheathing

Roof sheathing shall be in accordance with Tables 611.1-2 and 611.1-3 for wood structural panels, and Table 611.1-1 for lumber.

Joints in lumber sheathing shall occur over support unless approved end-matched is used, in which case each piece shall bear on at least two supports.

Wood structural panels used for roof sheathing shall be bonded by intermediate or exterior glue. Wood structural panel roof sheathing exposed on the underside shall be bonded with exterior glue.

620.9.11.9 Roof Planking

Planking shall be designed in accordance with the general provisions of this Chapter.

In lieu of such design, 50 mm tongue-and-groove planking may be used in accordance with Table 620.7-1. Joints in such planking may be randomly spaced, provided the system is applied to not less than three continuous spans, the planks are center-matched and end-matched or splined, each plank bears on at least one support, and the joints are separated by at least 600 mm in adjacent pieces.

620.10 Exit Facilities

In Seismic Zone 4, exterior exit balconies, stairs and similar exit facilities shall be positively anchored to the primary structure at not over 2.40 m. on center or shall be designed for lateral forces. Such attachment shall not be accomplished by use of toenails or nails subject to withdrawal.

SECTION 621 METAL PLATE-CONNECTED WOOD TRUSS DESIGN

621.1 Design and Fabrication

The design and fabrication of metal plate connected wood trusses shall be in accordance with ANSI/TPI 1-1995, National Design Standard for Metal Plate Connected Wood Truss Construction of the Truss Plate Institute.

621.2 Performance

Full-scale load tests in accordance with ANSI/TPI 2 may be required at the option of the building official to provide a means of demonstrating that minimum adequate performance is obtainable from specific metal plate connector plates, various lumber types and grades, a particular truss design and a particular fabrication procedure. ANSI/TPI 2 provides procedures for testing and evaluating wood trusses designed in accordance with ANSI/TPI 1.

621.3 In-Plant Inspection

Each truss manufacturer shall retain an approved agency having no financial interest in the plant being inspected to make nonscheduled inspections of truss fabrications and delivery operations. The inspections shall cover all phases of the truss operation, including lumber storage, handling, cutting, fixtures, presses or rollers, fabrication, bundling and banding handling, and delivery.

621.4 Marking

Each truss shall be legibly branded, marked or otherwise have permanently affixed thereto the following information located within 600 mm of the center of the span on the face of the bottom chord:

1. Identity of the company manufacturing the truss.
2. The design load.
3. The spacing of trusses.

SECTION 622 USE OF MACHINE-GRADED LUMBER (MGL)

622.1 General

In cases where the identification of a particular wood species is not known, therefore working stresses cannot be found in Table 615.2-1, machine graded lumber can be used for general and structural applications.

622.2 Design Properties for Machine-Graded Lumber

The design properties for machine graded lumber developed by the Forest Products Research and Development Institute are shown in Table 622.2-1 to 622.2-4. These properties are applicable for dry lumber (moisture content $\leq 16\%$) only. In green lumber (moisture content $\geq 28\%$), the design strength shall be reduced by 40% and modulus of elasticity by 20%. For lumber with moisture content between 16% and 28%, the design properties may be obtained by direct interpolation.

622.3 Design Using Machine-Graded Lumber

The reference values given in Section 622.2 may be used to design timber structures in accordance with the rules given by NSCP and other appropriate national and/or international standards.

622.4 Preservative Treatment

To ensure the durability of MGL against bio-deteriorating agents such as fungi and insects, MGL should be treated with an environment-friendly preservative.

622.5 Moisture Content

A given piece of lumber is considered dry, partially seasoned, and green, when their respective moisture contents is above 10%, 22 – 28%, and greater than 28%.

622.6 Markings

Prior to use, each machine graded lumber should be inspected for a mark that contains the mill in which the lumber was graded, organization that certifies the quality of the grading procedure, timber size, stress grade and moisture content.

Table 622.2-1 Reference Values for Dry Machine-Graded Lumber

Machine Stress Grade	Reference Values				Modulus of Elasticity, E (GPa)
	Bending, F_b (MPa)	Tension, F_t (MPa)	Compression, F_c (MPa)	Shear, F_v (MPa)	
	5	3	4	1.48	5.68
M5	5	3	4	1.64	8.57
M10	10	6	8	1.79	11.45
M15	15	9	12	1.95	14.34
M20	20	12	16	2.10	17.23
M25	25	15	20		

Table 622.2-2 Basic Working Loads for Nails in Lateral Loading (MGL)

Machine Stress Grade	Load Capacity (N/mm)						
	D*=2.5	D=2.8	D=3.15	D=3.75	D=4.5	D=5	D=5.6
M5	92	112	138	188	258	310	378
M10	136	166	204	276	380	457	557
M15	182	222	272	369	508	611	745
M20	229	279	279	466	641	771	940
M25	279	340	340	566	779	937	1143

*D=nail diameter (mm)

Table 622.2-3 Basic Working Loads for Nails in Withdrawal (MGL)

Machine Stress Grade	Load Capacity (N/mm)						
	D*=2.5	D=2.8	D=3.15	D=3.75	D=4.5	D=5	D=5.6
M5	47	52	59	70	84	94	105
M10	85	96	108	128	154	171	191
M15	134	150	168	201	241	267	300
M20	192	215	241	287	345	383	429
M25	259	290	327	389	466	518	581

*D=nail diameter (mm)

Table 622.2-4 Reference Values for Machine-Graded Lumber

Reference Values	Machine Stress Grade				
	M5	M10	M15	M20	M25
Bending, F_b (MPa)	5	10	15	20	25
Tension Parallel to grain, F_t (MPa)	3	6	9	12	15
Tension Perpendicular to grain, $F_{t\perp}$ (MPa)	0.29	0.29	0.29	0.29	0.30
Compression Parallel to grain, F_c (MPa)	4	8	12	16	20
Compression Perpendicular to grain, $F_{c\perp}$ (MPa)	2.3	3.3	4.3	5.2	6.2
Modulus of Elasticity (mean), E (GPa)	6.2	8.8	11.3	13.9	16.4
Modulus of Elasticity (20th percentile), E (GPa)	5.1	7.6	10.1	12.6	15.1
Shear Modulus, G_v (GPa)	0.39	0.52	0.65	0.78	0.91

Chapter 7

MASONRY

NATIONAL STRUCTURAL CODE OF THE PHILIPPINES VOLUME I BUILDINGS, TOWERS AND OTHER VERTICAL STRUCTURES

SEVENTH EDITION, 2015

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Table of Contents

SECTION 701 - GENERAL	7-4
701.1 Scope.....	7-4
701.2 Design Methods.....	7-4
701.3 Definitions.....	7-4
701.4 Notations	7-5
SECTION 702 - MATERIAL STANDARDS.....	7-7
702.1 Quality.....	7-7
702.2 Standards of Quality.....	7-7
SECTION 703 - MORTAR AND GROUT.....	7-9
703.1 General	7-9
703.2 Materials.....	7-9
703.3 Mortar.....	7-9
703.4 Grout	7-10
703.5 Additives and Admixtures.....	7-10
SECTION 704 - CONSTRUCTION	7-11
704.1 General	7-11
704.2 Materials, Handling, Storage and Preparation.....	7-11
704.3 Placing Masonry Units	7-11
704.4 Reinforcement Placing	7-11
704.5 Grouted Masonry.....	7-12
SECTION 705 - QUALITY ASSURANCE	7-14
705.1 General	7-14
705.2 Scope.....	7-14
705.3 Compliance with f_m'	7-14
705.4 Mortar Testing.....	7-16
705.5 Grout Testing.....	7-16
705.6 Recycled Aggregates.....	7-16
SECTION 706 - GENERAL DESIGN REQUIREMENTS	7-16
706.1 General.....	7-16
706.2 Allowable Stress Design and Strength Design Requirements for Unreinforced and Reinforced Masonry.....	7-20
706.3 Alternative Strength Design (ASD) and Strength Design Requirements for Reinforced Masonry.....	7-22
SECTION 707 - ALLOWABLE STRESS DESIGN (ASD) OF MASONRY	7-24
707.1 General.....	7-24
707.2 Design of Reinforced Masonry	7-27
707.3 Design of Unreinforced Masonry.....	7-31
SECTION 708 - STRENGTH DESIGN OF MASONRY.....	7-32
708.1 General	7-32
708.2 Reinforced Masonry.....	7-34

SECTION 709 - SEISMIC DESIGN	7-44
709.1 Scope	7-44
709.2 General	7-44
709.3 Seismic Performance Category A	7-44
709.4 Seismic Performance Category B	7-Error! Bookmark not defined.
709.5 Seismic Performance Category C	7-44
709.6 Seismic Performance Category D	7-45
709.7 Seismic Performance Category E	7-46
SECTION 710 - EMPIRICAL DESIGN OF MASONRY	7-47
710.1 Height	7-47
710.2 Lateral Stability	7-47
710.3 Compressive Stresses	7-47
710.4 Lateral Support	7-47
710.5 Minimum Thickness	7-49
710.6 Bond	7-50
710.7 Anchorage	7-50
710.8 Unburned Clay Masonry	7-51
710.9 Stone Masonry	7-51
SECTION 711 - GLASS MASONRY	7-52
711.1 General	7-52
711.2 Mortar Joints	7-52
711.3 Lateral Support	7-52
711.4 Reinforcement	7-52
711.5 Size of Panels	7-52
711.6 Expansion Joints	7-52
711.7 Reuse of Units	7-52
SECTION 712 - MASONRY FIREPLACES	7-55
712.1 Definition	7-55
712.2 Footings and Foundations	7-55
712.3 Seismic Reinforcing	7-55
712.4 Seismic Anchorage	7-55
712.5 Firebox Walls	7-55
712.6 Firebox Dimensions	7-55
712.7 Lintel and Throat	7-56
712.8 Smoke Chamber Walls	7-56
712.9 Hearth and Hearth Extension	7-56
712.10 Hearth Extension Dimensions	7-56
712.11 Fireplace Clearance	7-57
712.12 Fireplace Fireblocking	7-57
712.13 Exterior Air	7-57

SECTION 713 - MASONRY CHIMNEYS.....	7-58
713.1 Definition	7-58
713.2 Footings and Foundations	7-58
713.3 Seismic Reinforcing	7-58
713.4 Seismic Anchorage.....	7-59
713.5 Corbeling.....	7-59
713.6 Changes in Dimension.....	7-59
713.7 Offsets	7-59
713.8 Additional Load.....	7-59
713.9 Termination	7-59
713.10 Wall Thickness.....	7-59
713.11 Flue Lining (Material)	7-59
713.12 Clay Flue Lining (Installation)	7-61
713.13 Additional Requirements.....	7-61
713.14 Multiple Flues	7-61
713.15 Flue Area (Appliance).....	7-61
713.16 Flue Area (Masonry Fireplace)	7-61
713.17 Inlet	7-62
713.18 Masonry Chimney Cleanout Openings.....	7-62
713.19 Chimney Clearances.....	7-63
713.20 Chimney Fireblocking.....	7-63

SECTION 701 GENERAL

701.1 Scope

The materials, design, construction and quality assurance of masonry shall be in accordance with this chapter.

701.2 Design Methods

Masonry shall comply with the provisions of one of the following design methods in this chapter as well as the requirements of Sections 701 through 705.

701.2.1 Allowable Stress Design

Masonry designed by allowable stress design method shall comply with the provisions of Sections 706 and 707.

701.2.2 Strength Design

Masonry designed by the strength design method shall comply with the provisions of Sections 706 and 708.

701.2.3 Empirical Design

Masonry designed by the empirical design method shall comply with the provisions of Sections 706.1 and 710.

701.2.4 Glass Masonry

Glass masonry shall comply with the provisions of Section 711.

701.3 Definitions

For the purpose of this chapter, certain terms are defined as follows:

AREA, BEDDED is the area of the surface of a masonry unit which is in contact with mortar or the surface of another masonry unit in the plane of the joint.

AREA OF REINFORCEMENT, EFFECTIVE is the cross-sectional area of reinforcement multiplied by the cosine of the angle between the reinforcement and the direction for which effective area is to be determined.

AREA, GROSS is the total cross-sectional area of a specified section.

AREA, NET is the gross cross-sectional area minus the area of ungrouted cores, notches, cells and unbedded areas. Net area is the actual surface area of cross section of masonry.

AREA, TRANSFORMED is the equivalent area of one material to a second based on the ratio of modulus of elasticity of the first material to the second.

BOND, ADHESION is the adhesion between masonry units and mortar or grout.

BOND, REINFORCING is the adhesion between steel reinforcement and mortar or grout.

BOND BEAM is a horizontal grouted element within masonry in which reinforcement is embedded.

CELL is a void space having a gross cross-sectional area greater than 900 mm².

CLEANOUT is an opening to the bottom of a grout space of sufficient size and spacing to allow the removal of debris.

COLLAR JOINT is the mortared or grouted space between wythes of masonry.

COLUMN, REINFORCED is a vertical structural member in which both the reinforcement and masonry resist compression.

COLUMN, UNREINFORCED is a vertical structural member whose horizontal dimension measured at right angles to the thickness does not exceed three times the thickness.

DIMENSIONS, ACTUAL are the measured dimensions of a designated item. The actual dimension shall not vary from the specified dimension by more than the amount allowed in the appropriate standard of quality in Section 702.

DIMENSIONS, NOMINAL of masonry units are equal to its specified dimensions plus the thickness of the joint with which the unit is laid.

DIMENSIONS, SPECIFIED are the dimensions specified for the manufacture or construction of masonry, masonry units, joints or any other component of a structure.

GROUT LIFT is an increment of grout height within the total grout pour.

GROUT POUR is the total height of masonry wall to be grouted prior to the erection of additional masonry. A grout pour will consist of one or more grout lifts.

GROUTED HOLLOW-UNIT MASONRY is that form of grouted masonry construction in which certain

designated cells of hollow units are continuously filled with grout.

GROUTED MULTI-WYTHE MASONRY is that form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.

JOINT, BED is the joint with or without mortar that is horizontal at the time the masonry units are placed.

JOINT, HEAD is the joint with or without mortar having a vertical transverse plane.

MASONRY UNIT is brick, tile, stone, glass block or concrete block conforming to the requirements specified in Section 702.

HOLLOW-MASONRY UNIT is a masonry unit whose net cross-sectional areas (solid area) in any plane parallel to the surface containing cores, cells or deep frogs is less than 75 percent of its gross cross-sectional area measured in the same plane.

SOLID-MASONRY UNIT is a masonry unit whose net cross-sectional area in any plane parallel to the surface containing the cores or cells is at least 75 percent of the gross cross-sectional area measured in the same plane.

MORTARLESS MASONRY SYSTEM is a method of masonry wall construction that eliminates the use of mortar.

PRISM is an assemblage of masonry units and mortar (if present) with or without grout used as a test specimen for determining properties of the masonry.

REINFORCED MASONRY is that form of masonry construction in which reinforcement acting in conjunction with the masonry is used to resist forces.

SHELL is the outer portion of a hollow masonry unit as placed in masonry.

WALL, BONDED is a masonry wall in which two or more wythes are bonded to act as a structural unit.

WALL, CAVITY is a wall containing continuous air space with a minimum width of 50 mm and a maximum width of 100 mm between wythes which are tied with metal ties.

WALL TIE is a mechanical metal fastener which connects wythes of masonry to each other or to other materials.

WEB is an interior solid portion of a hollow-masonry unit as placed in masonry.

WYTHE is the portion of a wall which is one masonry unit in thickness. A collar joint is not considered a wythe.

701.4 Notations

A_b	= cross-sectional area of anchor bolt, mm ²
A_e	= effective area of masonry, mm ²
A_g	= gross area of wall, mm ²
A_{jh}	= total area of special horizontal reinforcement through wall frame joint, mm ²
A_{mv}	= net area of masonry section bounded by wall thickness and length of section in direction of shear force considered, mm ²
A_p	= area of tension (pullout) cone of embedded anchor bolt projected onto surface of masonry, mm ²
A_s	= effective cross-sectional area of reinforcement in column or flexural member, mm ²
A_{se}	= effective area of reinforcement,
A_{sh}	= total cross-sectional area of rectangular tie reinforcement for confined core, mm ²
A_v	= area of reinforcement required for shear reinforcement perpendicular to longitudinal reinforcement, mm ²
A'_s	= effective cross-sectional area of compression reinforcement in flexural member, mm ²
a	= depth of equivalent rectangular stress block, mm
a_b	= nominal balance depth of equivalent rectangular strength stress block, mm
B_{sn}	= nominal shear strength of anchor bolt, kN
B_t	= allowable tensile force on anchor bolt, kN
B_{tn}	= nominal tensile strength of anchor bolt, kN
B_v	= allowable shear force on anchor bolt, kN
b	= effective width of rectangular member or width of flange for T and I sections, mm
b_{su}	= factored shear force supported by anchor bolt, kN
b_t	= computed tensile force on anchor bolt, kN
b_{tu}	= factored tensile force supported by anchor bolt, kN
b_v	= computed shear force on anchor bolt, kN
b'	= width of web in T or I section, mm
C_d	= nominal shear strength coefficient as obtained from Table 708-2
c	= distance from neutral axis to extreme fiber, mm
D	= dead loads, or related internal moments and forces
d	= distance from compression face of flexural member to centroid of longitudinal tensile reinforcement, mm
d_b	= diameter of reinforcing bar, mm
d_{bb}	= diameter of largest beam longitudinal reinforcing bar passing through, or anchored in a joint, mm

d_{bp}	= diameter of largest beam longitudinal reinforcing bar passing through, or anchored in a joint, mm	I_g, I_{cr}	= gross, cracked moment of inertia of wall cross section, mm ⁴
E	= load effects of earthquake, or related internal moments and forces	j	= ratio or distance between centroid of flexural compressive forces and centroid of tensile forces of depth, d
E_m	= modulus of elasticity of masonry, MPa	K	= reinforcement cover or clear spacing, whichever is less, mm
e	= eccentricity of P_{uf} , mm	k	= ratio of depth of compressive stress in flexural member to depth, d
e_{mu}	= maximum usable compressive strain of masonry	L	= live loads, or related internal moments and forces
F	= loads due to weight and pressure of fluids or related moments and forces	L_w	= length of wall, mm
F_a	= allowable average axial compressive stress in columns for centroidally applied axial load only, MPa	l	= length of wall or segment, mm
F_b	= allowable flexural compressive stress in members subjected to bending load only, MPa	l_b	= embedment depth of anchor bolt, mm
F_{br}	= allowable bearing stress in masonry, MPa	l_{be}	= anchor bolt edge distance, the least distance measured from edge of masonry to surface of anchor bolt, mm
F_s	= allowable stress in reinforcement, MPa	l_d	= required development length of reinforcement, mm
F_{sc}	= allowable compressive stress in column reinforcement, MPa	M	= design moment, kNm
F_t	= allowable flexural tensile stress in masonry, MPa	M_a	= maximum moment in member at stage deflection is computed, kNm
F_v	= allowable shear stress in masonry, MPa	M_c	= moment capacity of compression reinforcement in flexural member about centroid of tensile force, kNm
f_a	= computed axial compressive stress due to design axial load, MPa	M_{cr}	= nominal cracking moment strength in masonry, kNm
f_b	= computed flexural stress in extreme fiber due to design bending loads only, MPa	M_m	= moment of compressive force in masonry about centroid of tensile force in reinforcement, kNm
f_{mb}	= computed compressive stress due to dead load only, MPa	M_n	= nominal moment strength, kNm
f_r	= modulus of rupture, MPa	M_s	= moment of tensile force in reinforcement about centroid of compressive force in masonry, kNm
f_s	= computed stress in reinforcement due to design loads, MPa	M_{ser}	= service moment at midheight of panel, including $P\Delta$ effects, kNm
f_v	= computed shear stress due to design load, MPa	M_u	= factored moment, kNm
f_y	= tensile yield stress of reinforcement, MPa	n	= modular ratio
f_{yh}	= tensile yield stress of horizontal reinforcement, MPa		= E_s/E_m
f'_g	= specified compressive strength of grout at age of 28 days, MPa	P	= design axial load, kN
f'_m	= specified compressive strength of masonry at age of 28 days, MPa	P_a	= allowable centroidal axial load for reinforced masonry columns, kN
G	= shear modulus of masonry, MPa	P_b	= nominal balanced design axial strength, kN
H	= loads due to weight and pressure of soil, water in soil or related internal moments and forces	P_f	= load from tributary floor or roof area, kN
h	= height of wall between points of support, mm	P_n	= nominal axial strength in masonry, kN
h_b	= beam depth, mm	P_o	= nominal axial load strength in masonry without flexure, kN
h_c	= cross-sectional dimension of grouted core measured center to center of confining reinforcement, mm	P_u	= factored axial load, kN
h_p	= pier depth in plane of wall frame, mm	P_{uf}	= factored load from tributary floor or roof loads, kN
h'	= effective height of wall or column, mm	P_{uw}	= factored weight of wall tributary to section under consideration, kN
I	= moment of inertia about neutral axis of cross-sectional area, mm ⁴	P_w	= weight of wall tributary to section under consideration, kN
I_e	= effective moment of inertia, mm ⁴	r	= radius of gyration (based on specified unit

	dimensions or Tables 711-1, 711-2 and 711-3), mm
r_b	= ratio of area of reinforcing bars cut off to total area of reinforcing bars at the section
S	= section modulus, mm ³
s	= spacing of stirrups or of bent bars in direction parallel to that of main reinforcement, mm
T	= effects of temperature, creep, shrinkage and differential settlement
t	= effective thickness of wythe, wall or column, mm
U	= required strength to resist factored loads, or related internal moments and forces
u	= bond stress per unit of surface area of reinforcing bar, MPa
V	= total design shear force, kN
V_{jh}	= total horizontal joint shear, kN
V_m	= nominal shear strength of masonry, kN
V_n	= nominal shear strength, kN
V_s	= nominal shear strength of shear reinforcement, kN
V_u	= required shear strength in masonry, kN
W	= wind load, or related internal moments in forces
w_u	= factored distributed lateral load
Δ_s	= horizontal deflection at mid height under factored load, mm
Δ_u	= deflection due to factored loads, mm
ρ	= ratio of area of flexural tensile reinforcement, A_s , to area b_d
ρ_b	= reinforcement ratio producing balanced strain conditions
ρ_n	= ratio of distributed shear reinforcement on plane perpendicular to plane of A_{mv}
Σ_o	= sum of perimeters of all longitudinal reinforcement, mm
$\sqrt{f'_m}$	= square root of specified strength of masonry at the age of 28 days, MPa
ϕ	= strength-reduction factor

SECTION 702 MATERIAL STANDARDS

702.1 Quality

Materials used in masonry shall conform to the requirements stated herein. If no requirements are specified in this section for a material, quality shall be based on generally accepted good practice, subject to the approval of the building official.

Reclaimed or previously used masonry units shall meet the applicable requirements as for new masonry units of the same material for their intended use.

702.2 Standards of Quality

The standards listed below labeled a "UBC 97 Standard" are also listed in Chapter 35, Part II of UBC, and are part of this code. The other standards listed below are recognized standards. See Sections 3503 and 3504 of UBC.

1. Aggregates

1.1 ASTM C144, Aggregates for Masonry Mortar

1.2 ASTM C404, Aggregates for Grout

2. Cement

2.1 ASTM C91-93, Cement, Masonry. (Plastic cement conforming to the requirements of UBC Standard 25-1 may be used in lieu of masonry cement when it also conforms to ASTM C 91-93).

2.2 ASTM C150, Portland Cement

2.3 ASTM C270, Mortar Cement

3. Lime

3.1 ASTM C5-79, Quicklime for Structural Purposes

3.2 ASTM C207-91, Hydrated Lime for Masonry Purposes. When Types N and NA hydrated lime are used in masonry mortar, they shall comply with the provisions of UBC Standard ASTM C270-95, Section 21.1506.7, excluding the plasticity requirement.

4. Masonry Units of Clay or Shale

- 4.1 ASTM C34, Structural Clay Load-bearing Wall Tile
- 4.2 ASTM C56, Structural Clay Nonload-bearing Tile
- 4.3 ASTM C62-87, Building Brick (solid units)
- 4.4 ASTM C126, Ceramic Glazed Structural Clay Facing Tile, Facing Brick and Solid Masonry Units. Load-bearing glazed brick shall conform to the weathering and structural requirements of ASTM C73-85, Section 21.106, Facing Brick
- 4.5 ASTM C216-86, Facing Brick (solid units)
- 4.6 ASTM C90-85, Hollow Brick
- 4.7 ASTM C67, Sampling and Testing Brick and Structural Clay Tile
- 4.8 ASTM C212, Structural Clay Facing Tile
- 4.9 ASTM C530, Structural Clay Non-Load bearing Screen Tile.

5. Masonry Units of Concrete

- 5.1 ASTM C55-85, Concrete Building Brick
- 5.2 ASTM C90-85, Hollow and Solid Load-bearing Concrete Masonry Units
- 5.3 ASTM C129-85, Non-load bearing Concrete Masonry Units
- 5.4 ASTM C140, Sampling and Testing Concrete Masonry Units
- 5.5 ASTM C426, Standard Test Method for Drying Shrinkage of Concrete Block

6. Masonry Units of Other Materials

- 6.1 Calcium silicate
- 6.2 ASTM C73-85, Calcium Silicate Face Brick (Sand-lime Brick)
- 6.3 ASTM C216, C62 or C652, Unburned Clay Masonry Units and Standard Methods of Sampling and ASTM C 67, Testing Unburned Clay Masonry Units

- 6.4 ACI-704, Cast Stone

- 6.5 ASTM E92b, Test Method for Compressive Strength of Masonry Prisms

7. Connectors

- 7.1 Wall ties and anchors made from steel wire shall conform to UBC Standard 21-10, Part II, and other steel wall ties and anchors shall conform to A36 in accordance with UBC Standard 22-1. Wall ties and anchors made from copper, brass or other nonferrous metal shall have minimum tensile yield strength of 200 MPa.
- 7.2 All such items not fully embedded in mortar or grout shall either be corrosion resistant or shall be coated after fabrication with copper, zinc or a metal having at least equivalent corrosion-resistant properties.

8. Mortar

- 8.1 ASTM C270-95, Mortar for Unit Masonry and Reinforced Masonry other than Gypsum
- 8.2 ASTM C270, Field Tests Specimens for Mortar
- 8.3 ASTM C780, Standard Test Method for Flexural Bond Strength of Mortar Cement

9. Grout

- 9.1 ASTM C1019-84, Method of Sampling and Testing Grout
- 9.2 ASTM C476-83, Grout for Masonry

10. Reinforcement

- 10.1 ASTM A82, Part I, Joint Reinforcement for Masonry
- 10.2 ASTM A615, A616, A617, A706, A767 and A775, Deformed and Plain Billet-steel Bars, Rail-steel Deformed and Plain Bars, Axle-steel Deformed and Plain Bars, and Deformed Low-alloy Bars for Concrete Reinforcement
- 10.3 ASTM A496, Part II, Cold-drawn Steel Wire for Concrete Reinforcement

SECTION 703

MORTAR AND GROUT

703.1 General

Mortar and grout shall comply with the provisions of this section. Special mortars, grouts or bonding systems may be used, subject to satisfactory evidence of their capabilities when approved by the building official.

703.2 Materials

Materials used as ingredients in mortar and grout shall conform to the applicable requirements in Section 702. Cementitious materials for grout shall be one or both of the following: lime and Portland cement. Cementitious materials for mortar shall be one or more of the following: lime, masonry cement, Portland cement and mortar cement. Cementitious materials or additives shall not contain epoxy resins and derivatives, phenols, asbestos fibers or fire clays. Water used in mortar or grout shall be clean and free of deleterious amounts of acid, alkalies or organic material or other harmful substances.

703.3 Mortar

703.3.1 General

Mortar shall consist of a mixture of cementitious materials and aggregate to which sufficient water and approved additives, if any, have been added to achieve a workable, plastic consistency.

703.3.2 Selecting Proportions

Mortar with specified proportions of ingredients that differ from the mortar proportions of Table 703-1 may be approved for use when it is demonstrated by laboratory or field experience that this mortar with the specified proportions of ingredients, when combined with the masonry units to be used in the structure, will achieve the specified compressive strength f'_m . Water content shall be adjusted to provide proper workability under existing field conditions. When the proportion of ingredients is not specified, the proportions by mortar type shall be used as given in Table 703-1.

Table 703-1 Mortar Proportions for Unit Masonry

MORTAR	TYPE	PROPORTIONS BY VOLUME (CEMENTITIOUS MATERIALS)							AGGREGATE MEASURED IN A DAMP, LOOSE CONDITION	
		Portland Cement or Blended Cement	Masonry Cement ¹			Mortar Cement ²				Hydrated Lime or Lime Putty
			M	S	N	M	S	N		
Cement - Lime	M	1	-	-	-	-	-	-	¼	Not less than 2 ¼ and not more than 3 times the sum of the separate volumes of cementitious materials
	S	1	-	-	-	-	-	-	over ¼ to ½	
	N	1	-	-	-	-	-	-	over ½ to 1 ¼	
	O	1	-	-	-	-	-	-	over 1 ¼ to 2 ½	
Mortar cement	M	1	-	-	-	-	-	1		
	M	-	-	-	-	1	-	-		
	S	½	-	-	-	-	-	1		
	S	-	-	-	-	-	1	-		
	N	-	-	-	-	-	-	1		
Masonry cement	M	1	-	-	1	-	-	-		
	M	-	1	-	-	-	-	-		
	S	½	-	-	1	-	-	-		
	S	-	-	1	-	-	-	-		
	N	-	-	-	1	-	-	-		
	O	-	-	-	1	-	-	-		

¹ Masonry cement conforming to the requirements of UBC Standard 21-11.

² Mortar cement conforming to the requirements of UBC Standard 21-14.

703.4 Grout**703.4.1 General**

Grout shall consist of a mixture of cementitious materials and aggregate to which water has been added such that the mixture will flow without segregation of the constituents. The specified compressive strength of grout, f'_g , shall not be less than 15 MPa

703.4.2 Selecting Proportions

Water content shall be adjusted to provide proper workability and to enable proper placement under existing field conditions, without segregation. Grout shall be specified by one of the following methods:

1. Proportions of ingredients and any additives shall be based on laboratory or field experience with the grout ingredients and the masonry units to be used.
2. The grout shall be specified by the proportion of its constituents in terms of parts by volume, or
3. Minimum compressive strength which will produce the required prism strength, or
4. Proportions by grout type shall be used as given in Table 703-2.

Table 703-2 Grout Proportions by Volume ¹

TYPE	PARTS BY VOLUME OF PORTLAND CEMENT OR BLENDED CEMENT	PARTS BY VOLUME OF HYDRATED LIME OR LIME PUTTY	AGGREGATE MEASURED IN A DAMP, LOOSE CONDITION	
			Fine	Coarse
Fine grout	1	0 to $\frac{1}{10}$	$2\frac{1}{4}$ to 3 times the sum of the volumes of the cementitious materials	-
Coarse grout	1	0 to $\frac{1}{10}$	$2\frac{1}{4}$ to 3 times the sum of the volumes of the cementitious materials	1 to 2 times the sum of the volumes of the cementitious materials

¹ Grout shall attain a minimum compressive strength at 28 days of 13.8 MPa. The building official may require a compressive field strength test of grout made in accordance with UBC Standard 21-18.

703.5 Additives and Admixtures**703.5.1 General**

Additives and admixtures to mortar or grout shall not be used unless approved by the building official.

703.5.2 Air Entrainment

Air-entraining substances shall not be used in mortar or grout unless tests are conducted to determine compliance with the requirements of this code.

703.5.3 Colors

Only pure mineral oxide, carbon black or synthetic colors may be used. Carbon black shall be limited to a maximum of 3 percent of the weight of the cement.

SECTION 704 CONSTRUCTION

704.1 General

Masonry shall be constructed according to the provision of this section.

704.2 Materials, Handling, Storage and Preparation

All materials shall comply with applicable requirements of Section 702. Storage, handling and preparation at the site shall conform also the following:

1. Masonry materials shall be stored so that at the time of use the materials are clean and structurally suitable for the intended use.
2. All metal reinforcement shall be free from loose rust and other coatings that would inhibit reinforcing bond.
3. At the time of laying, burned clay units and sand lime units shall have an initial rate of absorption not exceeding 1.6 liter per square meter during a period of one minute. In the absorption test, the surface of the unit shall be held 3 mm below the surface of the water.
4. Concrete masonry units shall not be wetted unless otherwise approved.
5. Materials shall be stored in a manner such that deterioration or intrusion of foreign materials is prevented and that the material will be capable of meeting applicable requirements at the time of mixing or placement.
6. The method of measuring materials for mortar and grout shall be such that proportions of the materials can be controlled.
7. Mortar or grout mixed at the job site shall be mixed for a period of time not less than three minutes or more than 10 minutes in a mechanical mixer with the amount of water required to provide the desired workability. Hand mixing of small amounts of mortar is permitted. Mortar may be re-tempered. Mortar or grout which has hardened or stiffened due to hydration of the cement shall not be used. In no case shall mortar be used two and one-half hours, nor grout used one and one half hours, after the initial mixing water has been added to the dry ingredients at the jobsite.

Exceptions:

Dry mixes for mortar and grout which are blended in the factory and mixed at the job site shall be mixed in mechanical mixers until workable, but not to exceed 10 minutes.

704.3 Placing Masonry Units

704.3.1 Mortar

The mortar, when used shall be sufficiently plastic and units shall be placed with sufficient pressure to extrude mortar from the joint and produce a tight joint. Deep furrowing which produces voids shall not be used.

When mortar is used, the initial bed joint thickness shall not be less than 6 mm or more than 25 mm; subsequent bed joints shall not be less than 6 mm or more than 16 mm in thickness.

704.3.2 Surfaces

Surfaces to be in contact with mortar or grout shall be clean and free of deleterious materials.

704.3.3 Solid Masonry Units

Solid masonry units shall have full head and bed joints.

704.3.4 Hollow Masonry Units

Except for mortarless system all head and bed joints shall be filled solidly with mortar for a distance in from the face of the unit not less than the thickness of the shell.

Head and bedded joints of open-ends units with beveled ends that are to be fully grouted need not be mortared. The beveled ends shall form a grout key which permits grout within 16 mm of the face of the unit. The units shall be tightly butted to prevent leakage of grout.

704.4 Reinforcement Placing

Reinforcement details shall conform to the requirements of this chapter. Metal reinforcement shall be located in accordance with the plans and specifications. Reinforcement shall be secured against displacement prior to grouting by wire positioners or other suitable devices at intervals not exceeding 200 bar diameters.

Tolerances for the placement of reinforcement in walls and flexural elements shall be plus or minus 12 mm for effective depth d equal to 200 mm or less, ± 25 mm for d equal to 600 mm or less but greater than 200 mm, and 32 mm for d greater than 600 mm. Tolerance for longitudinal location of reinforcement shall be ± 50 mm

704.5 Grouted Masonry

704.5.1 General Conditions

Grouted masonry shall be constructed in such a manner that all elements of the masonry act together as a structural element.

Prior to grouting, the grout space shall be clean so that all spaces to be filled with grout do not contain mortar projections greater than 12 mm, mortar droppings or other foreign material. Grout shall be placed so that all spaces designated to be grouted shall be filled with grout and the grout shall be confined to those specific spaces.

Grout materials and water content shall be controlled to provide adequate fluidity for placement without segregation of the constituents, and shall be mixed thoroughly.

The grouting of any section of wall shall be completed in one day with no interruptions greater than one hour.

Between grout pours, a horizontal construction joint shall be formed by stopping all wythes at the same elevation and with the grout stopping a minimum of 40 mm below a

mortar joint, except the top of the wall. Where bond beams occur, the grout pour shall be stopped a minimum of 10 mm below the top of the masonry.

Size and height limitations of the grout space or cell shall not be less than shown in Table 704-1. Higher grout pours or smaller cavity widths or cell size than shown in Table 704-1 may be used when approved, if it is demonstrated that grout spaces will be properly filled.

Cleanouts shall be provided for all grout pours over 1.50 m in height.

Where required, cleanouts shall be provided in the bottom course at every vertical bar but shall not be spaced more than 800 mm on center for solidly grouted masonry. When cleanouts are required, they shall be sealed after inspection and before grouting.

Where cleanouts are not provided, special provisions must be made to keep the bottom and sides of the grout spaces, as well as the minimum total clear area as required by Table 704-1, clean and clear prior to grouting.

Units may be laid to the full height of the grout pour and grout shall be placed in a continuous pour in grout lifts not exceeding 1.8 m. When approved, grout lifts may be greater than 1.8 m if it can be demonstrated the grout spaces can be properly filled.

All cells and spaces containing reinforcement shall be filled with grout.

Table 704-1 Grouting Limitations

GROUT TYPE	GROUT POUR MAXIMUM HEIGHT (m) ¹	MINIMUM DIMENSIONS OF THE TOTAL CLEAR AREAS WITHIN GROUT SPACES AND CELLS ^{2,3} (mm)	
		Multi-wythe Masonry	Hollow-unit Masonry
Fine	.30	20	35 x 50
Fine	1.50	35	35 x 50
Fine	2.40	35	35 x 50
Fine	3.60	35	45 x 75
Fine	7.20	50	75 x 75
Coarse	.30	35	35 x 75
Coarse	1.5	50	65 x 75
Coarse	2.40	50	75 x 75
Coarse	3.60	60	75 x 75
Coarse	7.20	75	75 x 100

¹ See also Section 2104.6.

² The actual grout space or grout cell dimensions must be larger than the sum of the following items (1) The required minimum dimensions of total clear areas in Table 704-1; (2) The width of any mortar projections within the space; and (3) The horizontal projections of the diameters of the horizontal reinforcing bars within a cross section of the grout space or cell.

³ The minimum dimensions of the total clear areas shall be made up of one or more open areas with at least one area being 19 mm or greater in width.

704.5.2 Construction Requirements

Reinforcement shall be placed prior to grouting. Bolts shall be accurately set with templates or by approved equivalent means and held in place to prevent dislocation during grouting.

Segregation of the grout materials and damage to the masonry shall be avoided during the grouting process.

Grout shall be consolidated by mechanical vibration during placement before loss of plasticity in a manner to fill the grout space. Grout pours greater than 300 mm in height shall be reconsolidated by mechanical vibration to minimize voids due to water loss. Grout pours 300 mm or less in height shall be mechanically vibrated or puddled.

In nonstructural elements which do not exceed 2.4 m in height above the highest point of lateral support, including fireplaces and residential chimneys, mortar of pouring consistency may be substituted for grout when the masonry is constructed and grouted in pours of 300 mm or less in height.

In multi-wythe grouted masonry, vertical barriers of masonry shall be built across the grout space the entire height of the grout pour and spaced not more than 9 m horizontally. The grouting of any section of wall between barriers shall be completed in one day with no interruption longer than one hour.

704.5.3 Aluminum Equipment

Grout shall not be handled nor pumped utilizing aluminum equipment unless it can be demonstrated with the materials and equipment to be used that there will be no deleterious effect on the strength of the grout.

704.5.4 Joint Reinforcement

Wire joint reinforcement used in the design as principal reinforcement in hollow-unit construction shall be continuous between supports unless splices are made by lapping:

1. Fifty wire diameters in a grouted cell, or
2. Seventy-five wire diameters in the mortared bed joint, or
3. In alternate bed joints of running bond masonry a distance not less than 50 diameters plus twice the spacing of the bed joints, or
4. As required by calculation and specific location in areas of minimum stress, such as points of inflection.

Side wires shall be deformed and shall conform to ASTM A82 Joint Reinforcement for Masonry.

SECTION 705

QUALITY ASSURANCE

705.1 General

Quality assurance shall be provided to ensure that materials, construction and workmanship are in compliance with the plans and specifications, and the applicable requirements of this chapter. When required, inspection records shall be maintained and made available to the building official.

705.2 Scope

Quality assurance shall include, but is not limited to, assurance that:

1. Masonry units, reinforcement, cement, lime, aggregate and all other materials meet the requirements of the applicable standards of quality and that they are properly stored and prepared for use.
2. Mortar and grout are properly mixed using specified proportions of ingredients. The method of measuring materials for mortar and grout shall be such that proportions of materials are controlled.
3. Construction details, procedures and workmanship are in accordance with the plans and specifications.
4. Placement, splices and reinforcement sizes are in accordance with the provisions of this chapter and the plans and specifications.

705.3 Compliance with f'_m

705.3.1 General

Compliance with the requirements for the specified compressive strength of masonry f'_m shall be in accordance with one of the sections in this subsection.

The actual compressive strength of masonry f'_m shall not be less than 4 MPa.

705.3.2 Masonry Prism Testing

The compressive strength of masonry determined in accordance with ASTM E447 for each set of prisms shall equal or exceed f'_m . Compressive strength of prisms shall be based on tests at 28 days. Compressive strength at seven days or three days may be used provided a relationship between seven-day and three-day and 28-day strength has been established for the project prior to the start of

construction. Verification by masonry prism testing shall meet the following:

1. A set of five masonry prisms shall be built and tested in accordance with ASTM E447 prior to the start of construction. Materials used for the construction of the prisms shall be taken from those specified to be used in the project. Prisms shall be constructed under the observation of the engineer-of-record or special inspector or an approved agency and tested by an approved agency.
2. When full allowable stresses are used in design, a set of three prisms shall be built and tested during construction in accordance with ASTM E447 for each 450 m² of wall area, but not less than one set of three masonry prisms for the project.
3. When one half the allowable masonry stresses are used in design, testing during construction is not required. A letter of certification from the manufacturer and/or supplier of the materials used to verify the f'_m in accordance with Section 705.3.2, Item 1, shall be provided at the time of, or prior to, delivery of the materials to the job site to ensure the materials used in construction are representative of the materials used to construct the prisms prior to construction.

705.3.3 Masonry Prism Test Record

Compressive strength verification by masonry prism test records shall meet the following:

1. A masonry prism test record approved by the building official of at least 30 masonry prisms which were built and tested in accordance with ASTM E447. Prisms shall have been constructed under the observation of an engineer or special inspector or an approved agency and shall have been tested by an approved agency.
2. Masonry prisms shall be representative of the corresponding construction.
3. The average compressive strength of the test record shall equal or exceed $1.33 f'_m$.
4. When full allowable stresses are used in design, a set of three masonry prisms shall be built during construction in accordance with ASTM E 447 for each 450 m² of wall area, but not less than one set of three prisms for the project.

Table 705-1 Specified Compressive Strength of Masonry, f'_m (MPa) Based on Specifying the Compressive Strength of Masonry Units

COMPRESSIVE STRENGTH OF CLAY MASONRY UNITS ^{1,2} (MPa)	SPECIFIED COMPRESSIVE STRENGTH OF MASONRY, f'_m	
	Type M or S Mortar ³ (MPa)	Type N Mortar ³ (MPa)
96.50 or more	36.50	30.30
82.70	32.40	26.20
68.90	27.60	22.70
55.10	23.10	18.60
41.30	18.60	15.20
27.60	13.80	11.00
COMPRESSIVE STRENGTH OF CONCRETE MASONRY UNITS ^{2,4} (MPa)	SPECIFIED COMPRESSIVE STRENGTH OF MASONRY, f'_m	
	Type M or S Mortar ³ (MPa)	Type N Mortar ³ (MPa)
33.10 or more	20.70	19.30
25.80	17.20	16.19
19.30	13.80	12.70
13.10	10.30	9.30
8.60	6.90	6.50

¹ Compressive strength of solid clay masonry units is based on gross area. Compressive strength of hollow clay masonry units is based on minimum net area. Values may be interpolated. When hollow clay masonry units are grouted, the grout shall conform to the proportion in Table 703-2.

² Assumed assemblage. The specified compressive strength of masonry f'_m is based on gross area strength when using solid units or solid grouted masonry and net area strength when using ungrouted hollow units.

³ Mortar for unit masonry, proportion specification, as specified in Table 703-1. These values apply to portland cement - lime mortars without added air - entraining materials.

⁴ Values may be interpolated. In grouted concrete masonry, the compressive strength of grout shall be equal to or greater than the compressive strength of the concrete masonry units.

- When one half the allowable masonry stresses are used in design, field testing during construction is not required. A letter of certification from the supplier of the materials to the job site shall be provided at the time of, or prior to, delivery of the materials to assure the materials used in construction are representative of the materials used to develop the prism test record in accordance with Section 705.3.3, Item 1.
- When one half the allowable masonry stresses are used in design, testing is not required for the units. A letter of certification from the manufacturer of the units shall be provided at the time of, or prior to, delivery of the units to the job site to assure the units comply with the compressive strength required in Table 705-1; and
- Mortar shall comply with the mortar type required in Table 705-1; and.

705.3.4 Unit Strength Method

Verification by the unit strength method shall meet the following:

- When full allowable stresses are used in design, units shall be tested prior to construction and test units during construction for each 450 m² of wall area for compressive strength to show compliance with the compressive strength required in Table 705-1; and
- When full stresses are used in design for concrete masonry, grout shall be tested for each 450 m² of wall area, but not less than one test per project, to show compliance with the compressive strength required in Table 705-1, Footnote 4.
- When one half the allowable stresses are used in design for concrete masonry, testing is not required for the grout. A letter of certification from the supplier of the grout shall be provided at the time of, or prior to, delivery of the grout to the job site to assure the grout complies with the compressive strength required in Table 705-1, Footnote 4; or
- When full allowable stresses are used in design for clay masonry, grout proportions shall be verified by the engineer-of-record or special inspector or an approved agency to conform with Table 703-2.

Exception:

Prior to the start of construction, prism testing may be used in lieu of testing the unit strength. During construction, prism testing may also be used in lieu of testing the unit strength and the grout as required by Section 705.3.4, Item 4.

7. When one half the allowable masonry stresses are used in design for clay masonry, a letter of certification from the supplier of the grout shall be provided at the time of, or prior to, delivery of the grout to the job site to assure the grout conforms to the proportions of Table 703-2.

705.3.5 Testing Prisms from Constructed Masonry

When approved by the building official, acceptance of masonry which does not meet the requirements of Section 705.3.2, 705.3.3 or 705.3.4 shall be permitted to be based on tests of prisms cut from the masonry construction in accordance with the following:

1. A set of three masonry prisms that are at least 28 days old shall be saw cut from the masonry for each 450 m² of the wall area that is in question but not less than one set of three masonry prisms for the project. The length, width and height dimensions of the prisms shall comply with the requirements of ASTM E 447. Transporting, preparation and testing of prisms shall be in accordance with ASTM E 447.
2. The compressive strength of prisms shall be the value calculated in accordance with UBC 97 Standard 21-17, Section 21.1707.2, except that the net cross-sectional area of the prism shall be based on the net mortar bedded area.
3. Compliance with the requirement for the specified compressive strength of masonry, f'_m , shall be considered satisfied provided the modified compressive strength equals or exceeds the specified f'_m . Additional testing of specimens cut from locations in question shall be permitted.

705.4 Mortar Testing

When required, mortar shall be tested in accordance with ASTM C 270.

705.5 Grout Testing

When required, grout shall be tested in accordance with ASTM C 476-83.

705.6 Recycled Aggregates

Recycled aggregates shall refer to those materials whose mixtures are part of masonry blocks or concrete debris that have been crushed for re-use. Recycled aggregates shall pass the necessary tests before considered for re-use.

SECTION 706 GENERAL DESIGN REQUIREMENTS

706.1 General

706.1.1 Scope

The design of masonry structures shall comply with the allowable stress design provisions of Section 707, or the strength design provisions of Section 708 or the empirical design provisions of Section 710, and with the provisions of this section. Unless otherwise stated, all calculations shall be made using or based on specified dimensions.

706.1.2 Plans

Plans submitted for approval shall describe the required design strengths of masonry materials and inspection requirements for which all parts of the structure were designed, and any load test requirements.

706.1.3 Design Loads

See Chapter 2 for design loads and load combinations.

706.1.4 Stack Bond

In bearing and nonbearing walls, except veneer walls, if less than 75 percent of the units in any transverse vertical plane lap the ends of the units below a distance less than one half the height of the unit, or less than one fourth the length of the unit, the wall shall be considered laid in stack bond.

706.1.5 Multi-wythe Walls

706.1.5.1 General

All wythes of multi-wythe walls shall be bonded by grout or tied together by corrosion-resistant wall ties or joint reinforcement conforming to the requirements of Section 702, and as set forth in this section.

706.1.5.2 Wall Ties in Cavity Wall Construction

Wall ties shall be of sufficient length to engage all wythes. The portion of the wall ties within the wythe shall be completely embedded in mortar or grout. The ends of the wall ties shall be bent to 90-degree angles with an extension not less than 50 mm long. Wall ties not completely embedded in mortar or grout between wythes shall be a single piece with each end engaged in each wythe.

There shall be at least one $\phi 10$ mm wall tie for each 0.40 m^2 of wall area. For cavity walls in which the width of the cavity is greater than 75 mm, but not more than 115 mm, at least one 10 mm diameter wall tie for each 0.25 m^2 of wall area shall be provided.

Ties in alternate courses shall be staggered. The maximum vertical distance between ties shall not exceed 600 mm and the maximum horizontal distance between ties shall not exceed 900 mm.

Additional ties spaced not more than 900 mm apart shall be provided around openings within a distance of 300 mm from the edge of the opening.

Adjustable wall ties shall meet the following requirements:

1. One tie shall be provided for each 0.16 m^2 of wall area. Horizontal and vertical spacing shall not exceed 400 mm. Maximum misalignment of bed joints from one wythe to the other shall be 30 mm.
2. Maximum clearance between the connecting parts of the tie shall be 1.5 mm. When used, pintle ties shall have at least two 5 mm diameter pintle legs.

Wall ties of different size and spacing that provide equivalent strength between wythes may be used.

706.1.5.3 Wall Ties for Grouted Multi-wythe Construction

Wythes of multi-wythe walls shall be bonded together with at least 4.8 mm diameter steel wall tie for each 0.20 m^2 of area. Wall ties of different size and spacing that provide equivalent strength between wythes may be used.

706.1.5.4 Joint Reinforcement

Prefabricated joint reinforcement for masonry wall shall have at least one cross wire of at least No. 9 gage steel for each 0.20 m^2 of wall area. The vertical spacing of the joint reinforcement shall not exceed 400 mm. The longitudinal wires shall be thoroughly embedded in the bed joint mortar. The joint reinforcement shall engage all wythes.

Where the space between tied wythes is solidly filled with grout or mortar, the allowable stresses and other provisions for masonry bonded walls shall apply. Where the space is not filled, tied walls shall conform to the allowable stress, lateral support, thickness (excluding cavity), height and tie requirements for cavity walls.

706.1.6 Vertical Support

Structural members providing vertical support of masonry shall provide a bearing surface on which the initial bed joint shall not be less than 6 mm or more than 25 mm in thickness and shall be of noncombustible material, except where masonry is a nonstructural decorative feature or wearing surface.

706.1.7 Lateral Support

Lateral support of masonry may be provided by cross walls, columns, pilasters, counterforts or buttresses where spanning horizontally or by floors, beams, girts or roofs where spanning vertically.

The clear distance between lateral supports of a beam shall not exceed 32 times the least width of the compression area.

706.1.8 Protection of Ties and Joint Reinforcement

A minimum of 16 mm mortar cover shall be provided between ties or joint reinforcement and any exposed face. The thickness of grout or mortar between masonry units and joint reinforcement shall not be less than 6 mm, except that 6 mm diameter reinforcement or bolts may be placed in bed joints which are at least twice the thickness of the reinforcement or bolts.

706.1.9 Pipes and Conduits Embedded in Masonry

Pipes or conduit shall not be embedded in any masonry in a manner that will reduce the capacity of the masonry to less than that necessary for required strength or required fire protection.

Placement of pipes or conduits in unfilled cores of hollow-unit masonry shall not be considered as embedment.

Exceptions:

1. *Rigid electric conduits may be embedded in structural masonry when their locations have been detailed on the approved plan.*
2. *Any pipe or conduit may pass vertically or horizontally through any masonry by means of a sleeve at least large enough to pass any hub or coupling on the pipeline. Such sleeves shall not be placed closer than three diameters, center to center, nor shall they unduly impair the strength of construction.*

706.1.10 Load Test

When a load test is required, the member or portion of the structure under consideration shall be subjected to a superimposed load equal to twice the design live load plus one half of the dead load: $0.5 D + 2 L$

This load shall be left in position for a period of 24 hours before removal. If, during the test or upon removal of the load, the member or portion of the structure shows evidence of failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or where approved, a lower rating shall be established.

A flexural member shall be considered to have passed the test if the maximum deflection D at the end of the 24-hour period does not exceed the value of Eq. 706-1 or 706-2 and the beams and slabs show a recovery of at least 75 percent of the observed deflection within 24 hours after removal of the load.

$$D = \frac{\ell}{200} \quad (706-1)$$

$$D = \frac{\ell^2}{4,000t} \quad (706-2)$$

706.1.11 Reuse of Masonry Units

Masonry units may be reused when clean, whole and conforming to the other requirements of this section. All structural properties of masonry of reclaimed units shall be determined by approved test.

706.1.12 Special Provisions in Area of Seismic Risk**706.1.12.1 General**

Masonry structures constructed in the seismic zones shown in Figure 208-1 shall be designed in accordance with the design requirements of this chapter and the special provisions for each seismic zone given in this section.

706.1.12.2 Special Provisions for Seismic Zone 2

Masonry structures in Seismic Zone 2 shall comply with the following special provisions:

1. Columns shall be reinforced as specified in Sections 706.3.6, 706.3.7 and 707.2.13.
2. Vertical wall reinforcement of at least 130 mm^2 in cross-sectional area shall be provided continuously from support to support at each corner, at each side of each opening, at the ends of walls and at maximum spacing of 1.20 m apart horizontally throughout walls.
3. Horizontal wall reinforcement not less than 130 mm^2 in cross-sectional area shall be provided (1) at the bottom and top of wall openings and shall extend not less than 600 mm or less than 40 bar diameters past the opening, (2) continuously at structurally connected roof and floor levels and at the top of walls, (3) at the bottom of walls or in the top of foundations when doweled in walls, and (4) at maximum spacing of 3.0 m unless uniformly distributed joint reinforcement is provided. Reinforcement at the top and bottom of openings when continuous in walls may be used in determining the maximum spacing specified in Item 1 of this paragraph.
4. Where stack bond is used, the minimum horizontal reinforcement ratio shall be **0.0007 *bt***. This ratio shall be satisfied by uniformly distributed joint reinforcement or by horizontal reinforcement spaced not over 1.2 m and fully embedded in grout or mortar.
5. The following materials shall not be used as part of the vertical or lateral load-resisting system: Type O mortar, masonry cement, plastic cement, non-load bearing masonry units and glass block.

706.1.12.3 Special Provisions for Seismic Zone 4

All masonry structures built in Seismic Zone 4 shall be designed and constructed in accordance with requirements for Seismic Zone 2 and with the following additional requirements and limitations:

1. Column Reinforcement Ties

In columns that are stressed by tensile or compressive axial overturning forces from seismic loading, the spacing of column ties shall not exceed 200 mm for the full height of such columns.

In all other columns, ties shall be spaced a maximum of 0.20 m in the tops and bottoms of the columns for a distance of the greatest among (1) one sixth of the clear column height, (2) 450 mm, or (3) the maximum column cross-sectional dimension.

Tie spacing for the remaining column height shall not exceed the lesser of 16 bar diameters, 48 tie diameters, the least column cross-sectional dimension, or 450 mm.

Column ties shall terminate with a minimum 135° hook with extensions not less than six bar diameters or 100 mm. Such extensions shall engage the longitudinal column reinforcement and project into the interior of the column. Hooks shall comply with Section 707.2.2.5, Item 3.

Exceptions:

Where ties are placed in horizontal bed joints, hooks shall consist of a 90-degree bend having an inside radius of not less than four tie diameters plus an extension of 32 tie diameters.

2. Shear Walls

2.1 Reinforcement

The portion of the reinforcement required to resist shear shall be uniformly distributed and shall be joint reinforcement, deformed bars or a combination thereof. The spacing of reinforcement in each direction shall not exceed one half the length of the element, nor one half the height of the element, nor 1.2 m.

Joint reinforcement used in exterior walls and considered in the determination of the shear strength of the member shall be hot-dipped galvanized in accordance with ASTM A 385 & A 641.

Reinforcement required to resist in-plane shear shall be terminated with a standard hook as defined in Section 707.2.2.5 or with an extension of proper embedment length beyond the reinforcement at the end of the wall section. The hook or extension may be turned up, down or horizontally. Provisions shall be made not to obstruct grout placement. Wall reinforcement terminating in columns or beams shall be fully anchored into these elements.

2.2 Bond

Multi-wythe grouted masonry shear walls shall be designed with consideration of the adhesion bond strength between the grout and masonry units. When bond strengths are not known from previous tests, the bond strength shall be determined by tests.

2.3 Wall Reinforcement

All walls shall be reinforced with both vertical and horizontal reinforcement. The sum of the areas of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall, and the minimum area of reinforcement in either direction shall not be less than 0.001 times the gross cross-sectional area of the wall. The minimum steel requirements for Seismic Zone 2 in Section 706.1.12.2, Items 2 and 3, may be included in the sum. The spacing of reinforcement shall not exceed 1.2 m. The diameter of reinforcement shall not be less than 10 mm except that joint reinforcement may be considered as a part or all of the requirement for minimum reinforcement. Reinforcement shall be continuous around wall corners and through intersections. Only reinforcement which is continuous in the wall or element shall be considered in computing the minimum area of reinforcement. Reinforcement with splices conforming to Section 707.2.2.6 shall be considered as continuous reinforcement.

2.4 Stack Bond

Where stack bond is used, the minimum horizontal reinforcement ratio shall be **0.0015 *bt***. Where open-end units are used and grouted solid, the minimum horizontal reinforcement ratio shall be **0.001 *bt***.

Reinforced hollow-unit stacked bond construction which is part of the seismic-resisting system shall use open-end units so that all head joints are made solid, shall use bond beam units to facilitate the flow of grout and shall be grouted solid.

3. Type N Mortar

Type N mortar shall not be used as part of the vertical- or lateral-load-resisting system.

4. Concrete Abutting Structural Masonry

Concrete abutting structural masonry, such as at starter courses or at wall intersections not designed as true separation joints, shall be roughened to a full amplitude of 1.5 mm and shall be bonded to the masonry in accordance with the requirements of this chapter as if it were masonry. Unless keys or proper reinforcement is provided, vertical joints as specified in Section 706.1.4 shall be considered to be stack bond and the reinforcement as required for stack bond shall extend through the joint and be anchored into the concrete.

706.2 Allowable Stress Design and Strength Design Requirements for Unreinforced and Reinforced Masonry

706.2.1 General

In addition to the requirements of Section 706.1, the design of masonry structures by the allowable stress design method and strength design method shall comply with the requirements of this section. Additionally, the design of reinforced masonry structures by these design methods shall comply with the requirements of Section 706.3.

706.2.2 Specified Compressive Strength of Masonry

The allowable stresses for the design of masonry shall be based on value of f'_m selected for the construction.

Verification of the value of f'_m shall be based on compliance with Section 705.3. Unless otherwise specified, f'_m shall be based on 28-day tests. If other than a 28-day test age is used, the value of f'_m shall be as indicated in design drawings or specifications. Design drawings shall show the value of f'_m for which each part of the structure is designed.

706.2.3 Effective Thickness

706.2.3.1 Single-Wythe Walls

The effective thickness of single-wythe walls of either solid or hollow units is the specified thickness of the wall.

706.2.3.2 Multi-wythe Walls

The effective thickness of multi-wythe walls is the specified thickness of the wall if the space between wythes is filled with mortar or grout. For walls with an open space between wythes, the effective thickness shall be determined as for cavity walls.

706.2.3.3 Cavity Walls

Where both wythes of a cavity wall are axially loaded, each wythe shall be considered to act independently and the effective thickness of each wythe is as defined in Section 706.2.3.1. Where only one wythe is axially loaded, the effective thickness of the cavity wall is taken as the square root of the sum of the squares of the specified thicknesses of the wythes.

Where a cavity wall is composed of a single wythe and a multi-wythe, and both sides are axially loaded, each side of the cavity wall shall be considered to act independently and the effective thickness of each side is as defined in Sections 706.2.3.1 and 706.2.3.2. Where only one side is axially loaded, the effective thickness of the cavity wall is the square root of the sum of the squares of the specified thicknesses of the sides.

706.2.3.4 Columns

The effective thickness for rectangular columns in the direction considered is the specified thickness. The effective thickness for non-rectangular columns is the thickness of the square column with the same moment of inertia about its axis as that about the axis considered in the actual column.

706.2.4 Effective Height

The effective height of columns and walls shall be taken as the clear height of members laterally supported at the top and bottom in a direction normal to the member axis considered. For members not supported at the top normal to the axis considered, the effective height is twice the height of the member above the support. Effective height less than clear height may be used if justified.

706.2.5 Effective Area

The effective cross-sectional area shall be based on the minimum bedded area of hollow units, or the gross area of solid units plus any grouted area. Where hollow units are used with cells perpendicular to the direction of stress, the effective area shall be the lesser of the minimum bedded area or the minimum cross-sectional area. Where bed joints are raked, the effective area shall be correspondingly reduced. Effective areas for cavity walls shall be that of the loaded wythes.

706.2.6 Effective Width of Intersecting Walls

Where a shear wall is anchored to an intersecting wall or walls, the width of the overhanging flange formed by the intersected wall on either side of the shear wall, which may be assumed working with the shear wall for purposes of flexural stiffness calculations, shall not exceed six times the thickness of the intersected wall. Limits of the effective flange may be waived if justified. Only the effective area of the wall parallel to the shear forces may be assumed to carry horizontal shear.

706.2.7 Distribution of Concentrated Vertical Loads in Walls

The length of wall laid up in running bond which may be considered capable of working at the maximum allowable compressive stress to resist vertical concentrated loads shall not exceed the center-to-center distance between such loads, nor the width of bearing area plus four times the wall thickness. Concentrated vertical loads shall not be assumed to be distributed across continuous vertical mortar or control joints unless elements designed to distribute the concentrated vertical loads are employed.

706.2.8 Loads on Nonbearing Walls

Masonry walls used as interior partitions or as exterior surfaces of a building which do not carry vertical loads imposed by other elements of the building shall be designed to carry their own weight plus any superimposed finish and lateral forces. Bonding or anchorage of nonbearing walls shall be adequate to support the walls and to transfer lateral forces to the supporting elements.

706.2.9 Vertical Deflection

Elements supporting masonry shall be designed so that their vertical deflection will not exceed 1/600 of the clear span under total loads. Lintels shall bear on supporting masonry on each end such that allowable stresses in the supporting masonry are not exceeded. A minimum bearing length of 100 mm shall be provided for lintels bearing on masonry.

706.2.10 Structural Continuity

Intersecting structural elements intended to act as a unit shall be anchored together to resist the design forces.

706.2.11 Walls Intersecting with Floors and Roofs

Walls shall be anchored to all floors, roofs or other elements which provide lateral support for the wall. Where floors or roofs are designed to transmit horizontal forces to walls, the anchorage to such walls shall be designed to resist the horizontal force.

706.2.12 Modulus of Elasticity of Materials**706.2.12.1 Modulus of Elasticity of Masonry**

The moduli for masonry may be estimated as provided below. Actual values, where required, shall be established by test. The modulus of elasticity of masonry shall be determined by the secant method in which the slope of the line for the modulus of elasticity is taken from $0.05 f'_m$ to a point on the curve at $0.33 f'_m$. These values are not to be reduced by one half as set forth in Section 707.1.2.

Modulus of elasticity of clay or shale unit masonry.

$$E_m = 750 f'_m, \quad 20.5 \text{ GPa maximum} \quad (706-3)$$

Modulus of elasticity of concrete unit masonry.

$$E_m = 750 f'_m, \quad 20.5 \text{ GPa maximum} \quad (706-4)$$

706.2.12.2 Modulus of Elasticity of Steel

$$E_s = 200 \text{ GPa} \quad (706-5)$$

706.2.13 Shear Modulus of Masonry

$$G = 0.4 E_m \quad (706-6)$$

706.2.14 Placement of Embedded Anchor Bolts**706.2.14.1 General**

Placement requirements for plate anchor bolts, headed anchor bolts and bent bar anchor bolts shall be determined in accordance with this subsection. Bent bar anchor bolts shall have a hook with a 90-degree bend with an inside diameter of three bolt diameters, plus an extension of one and one half bolt diameters at the free end. Plate anchor bolts shall have a plate welded to the shank to provide anchorage equivalent to headed anchor bolts.

The effective embedment depth l_b for plate or headed anchor bolts shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the plate or head of the anchorage, and l_b for bent bar anchors shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the bent end minus one anchor bolt diameter. All bolts shall be grouted in place with at least 25 mm of grout between the bolt and the masonry, except that 6 mm bolts may be placed in bed joints which are at least 12 mm in thickness.

706.2.14.2 Minimum Edge Distance

The minimum anchor bolt edge distance l_{be} measured from the edge of the masonry parallel with the anchor bolt to the surface of the anchor bolt shall be 38 mm.

706.2.14.3 Minimum Embedment Depth

The minimum embedment depth of anchor bolts l_b shall be four bolt diameters but not less than 50 mm.

706.2.14.4 Minimum Spacing between Bolts

The minimum center-to-center distance between anchor bolts shall be four bolt diameters.

706.2.15 Flexural Resistance of Cavity Walls

For computing the flexural resistance of cavity walls, lateral loads perpendicular to the plane of the wall shall be distributed to the wythes according to their respective flexural rigidities.

706.3 Alternative Strength Design (ASD) and Strength Design Requirements for Reinforced Masonry

706.3.1 General

In addition to the requirements of Sections 706.1 and 706.2, the design of reinforced masonry structures by the working stress design method or the strength design method shall comply with the requirements of this section.

706.3.2 Plain Bars

The use of plain bars larger than 6 mm in diameter is not permitted.

706.3.3 Spacing of Longitudinal Reinforcement

The clear distance between parallel bars, except in columns, shall not be less than the nominal diameter of the bars or 25 mm, except that bars in a splice may be in contact. This clear distance requirement applies to the clear distance between a contact splice and adjacent splices or bars.

The clear distance between the surface of a bar and any surface of a masonry unit shall not be less than 6 mm for fine grout and 12 mm for coarse grout. Cross webs of hollow units may be used as support for horizontal reinforcement.

706.3.4 Anchorage of Flexural Reinforcement

The tension or compression in any bar at any section shall be developed on each side of that section by the required development length. The development length of the bar may be achieved by a combination of an embedment length, anchorage or, for tension only, hooks.

Except at supports or at the free end of cantilevers, every reinforcing bar shall be extended beyond the point at which it is no longer needed to resist tensile stress for a distance equal to 12 bar diameters or the depth of the beam, whichever is greater. No flexural bar shall be terminated in a tensile zone unless at least one of the following conditions l_b . The shear is not over one half that permitted, including allowance for shear reinforcement where provided.

1. Additional shear reinforcement in excess of that required is provided each way from the cutoff a distance equal to the depth of the beam. The shear reinforcement spacing shall not exceed $d/8r_b$.
2. The continuing bars provide double the area required for flexure at that point or double the perimeter required for reinforcing bond.

At least one third of the total reinforcement provided for negative moment at the support shall be extended beyond the extreme position of the point of inflection a distance sufficient to develop one half the allowable stress in the bar, not less than 1/16 of the clear span, or the depth d of the member, whichever is greater.

Tensile reinforcement for negative moment in any span of a continuous restrained or cantilever beam, or in any member of a rigid frame, shall be adequately anchored by reinforcement bond, hooks or mechanical anchors in or through the supporting member.

At least one third of the required positive moment reinforcement in simple beams or at the freely supported end of continuous beams shall extend along the same face of the beam into the support at least 150 mm. At least one fourth of the required positive moment reinforcement at the continuous end of continuous beams shall extend along the same face of the beam into the support at least 150 mm.

Compression reinforcement in flexural members shall be anchored by ties or stirrups not less than 6 mm in diameter, spaced not farther apart than 16 bar diameters or 48 tie diameters, whichever is less. Such ties or stirrups shall be used throughout the distance where compression reinforcement is required.

706.3.5 Anchorage of Shear Reinforcement.

Single, separate bars used as shear reinforcement shall be anchored at each end by one of the following methods:

1. Hooking tightly around the longitudinal reinforcement through 180 degrees.
2. Embedment above or below the mid-depth of the beam on the compression side a distance sufficient to develop the stress in the bar for plain or deformed bars.
3. By a standard hook, as defined in Section 707.2.2.5, considered as developing 50 MPa, plus embedment sufficient to develop the remainder of the stress to which the bar is subjected. The effective embedded length shall not be assumed to exceed the distance between the mid-depth of the beam and the tangent of the hook.

The ends of bars forming a single U or multiple U stirrup shall be anchored by one of the methods set forth in Items 1 through 3 above or shall be bent through an angle of at least 90 degrees tightly around a longitudinal reinforcing bar not less in diameter than the stirrup bar, and shall project beyond the bend at least 12 stirrup diameters.

The loops or closed ends of simple U or multiple U stirrups shall be anchored by bending around the longitudinal reinforcement through an angle of at least 90 degrees and project beyond the end of the bend at least 12 stirrup diameters.

706.3.6 Lateral Ties

All longitudinal bars for columns shall be enclosed by lateral ties. Lateral support shall be provided to the longitudinal bars by the corner of a complete tie having an included angle of not more than 135 degrees or by a standard hook at the end of a tie. The corner bars shall have such support provided by a complete tie enclosing the longitudinal bars. Alternate longitudinal bars shall have such lateral support provided by ties and no bar shall be farther than 150 mm from such laterally supported bar.

Lateral ties and longitudinal bars shall be placed not less than 38 mm and not more than 125 mm from the surface of the column. Lateral ties may be placed against the longitudinal bars or placed in the horizontal bed joints where the requirements of Section 706.1.8 are met. Spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie diameters or the least dimension of the column but not more than 450 mm.

Ties shall be at least 6 mm in diameter for 20 mm or smaller longitudinal bars and at least 10 mm for longitudinal bars larger than 20 mm. Ties smaller than 10 mm may be used for longitudinal bars larger than 20 mm, provided the total cross-sectional area of such smaller ties crossing a longitudinal plane is equal to that of the larger ties at their required spacing.

706.3.7 Column Anchor Bolt Ties

Additional ties shall be provided around anchor bolts which are set in the top of columns. Such ties shall engage at least four bolts or, alternately, at least four vertical column bars or a combination of bolts and bars totaling at least four. Such ties shall be located within the top 125 mm of the column and shall provide a total of 260 mm² or more in cross-sectional area. The uppermost tie shall be within 50 mm of the top of the column.

706.3.8 Effective Width of Compression Area

In computing flexural stresses in walls where reinforcement occurs, the effective width assumed for running bond masonry shall not exceed six times the nominal wall thickness or the center-to-center distance between reinforcement. Where stack bond is used, the effective width shall not exceed three times the nominal wall thickness or the center-to-center distance between reinforcement or the length of one unit, unless solid grouted open-end units are used.

SECTION 707

ALLOWABLE STRESS DESIGN (ASD) OF MASONRY

707.1 General

707.1.1 Scope

The design of masonry structures using allowable stress design shall comply with the provisions of Section 706 and this section. Stresses in clay or concrete masonry under service loads shall not exceed the values given in this section.

707.1.2 Allowable Masonry Stresses

When quality assurance provisions do not include requirements for special inspection as prescribed in Section 701, the allowable stresses for masonry in Section 707 shall be reduced by one half.

When one half allowable masonry stresses are used in Seismic Zone 4, the value of f'_m from Table 705-1 shall be limited to a maximum of 10 MPa for concrete masonry and 18 MPa for clay masonry unless the value of f'_m is verified by tests in accordance with Section 705.3.4, Items 1 and 4 or 6. A letter of certification is not required.

When one half allowable masonry stresses are used for design in Seismic Zones 4, the value of f'_m shall be limited to 10 MPa for concrete masonry and 18 MPa for clay masonry for Section 705.3.2, Item 3, and Section 705.3.3, Item 5, unless the value of f'_m is verified during construction by the testing requirements of Section 705.3.2, Item 2. A letter of certification is not required.

707.1.3 Minimum Dimensions for Masonry Structures Located in Seismic Zones 2 and 4

Elements of masonry structures located in Seismic Zones 2 and 4 shall be in accordance with this section.

707.1.3.1 Bearing Walls

The nominal thickness of reinforced masonry bearing walls shall not be less than 150 mm except that nominal 100 mm load-bearing reinforced hollow-clay unit masonry walls may be used, provided net area unit strength exceeds 55 MPa, units are laid in running bond, bar sizes do not exceed 12 mm with no more than two bars or one splice in a cell, and joints are flush cut, concave or a protruding V section.

707.1.3.2 Columns

The least nominal dimension of a reinforced masonry column shall be 300 mm except that, for ASD, if the allowable stresses are reduced by one half, the minimum nominal dimension shall be 200 mm.

707.1.4 Design Assumptions

The working stress design procedure is based on working stresses and linear stress-strain distribution assumptions with all stresses in the elastic range as follows:

1. Plane sections before bending remain plane after bending.
2. Stress is proportional to strain.
3. Masonry elements combine to form a homogenous member.

707.1.5 Embedded Anchor Bolts

707.1.5.1 General

Allowable loads for plate anchor bolts, headed anchor bolts and bent bar anchor bolts shall be determined in accordance with this section.

707.1.5.2 Tension

Allowable loads in tension shall be the lesser value selected from Table 707-1 and 707-2 or shall be determined from the lesser of Eq. 707-1 or Eq. 707-2.

$$B_t = 0.042A_p\sqrt{f'_m} \quad (707-1)$$

$$B_t = 0.2bf_y \quad (707-2)$$

The area A_p shall be the lesser of Eq. 707-3 or Eq. 707-4 and where the projected areas of adjacent anchor bolts overlap <overlap>, A_p of each anchor bolt shall be reduced by one half of the overlapping area.

$$A_p = \pi l_b^2 \quad (707-3)$$

$$A_p = \pi l_{be}^2 \quad (707-4)$$

Table 707-1 Allowable Tension, B_t , for Embedded Anchor Bolts for Clay and Concrete Masonry, kN^{1,2,3}

f'_m (MPa)	EMBEDMENT LENGTH, l_b , or EDGE DISTANCE, l_{be} (mm)						
	50	75	100	125	150	200	250
10.3	1.10	2.45	4.32	6.76	9.74	17.3	27.0
12.4	1.20	2.67	4.76	7.43	10.7	18.9	29.6
13.8	1.25	2.80	4.98	7.83	11.2	20.0	31.2
17.2	1.38	3.16	5.61	8.72	12.6	22.4	34.9
20.7	1.50	3.43	6.14	9.57	13.8	24.5	38.3
27.6	1.78	3.96	7.08	11.04	15.9	28.3	44.2
34.4	1.96	4.45	7.92	12.37	17.8	31.6	49.4
41.3	2.15	4.85	8.68	13.53	19.5	34.7	54.3

¹ The allowable tension values in Table 707-1 are based on compressive strength of masonry assemblages. Where yield strength of anchor bolt steel governs, the allowable tension in kN is given in Table 707-2.

² Values are for bolts of at least A 307 quality. Bolts shall be those specified in Section 706.2.14.1

³ Values shown are for work with or without special inspection.

Table 707-2 Allowable Tension, B_t , for Embedded Anchor Bolts for Clay and Concrete Masonry, kN^{1,2}

ANCHOR BOLT DIAMETER (mm)							
6	10	12	16	20	22	25	28
1.56	3.51	6.27	9.83	14.1	19.3	25.1	31.9

¹ Values are for bolts of at least A 307 quality. Bolts shall be those specified in Section 706.2.14.1

² Values shown are for work with or without special inspection.

707.1.5.3 Shear

Allowable loads in shear shall be the value selected from Table 707-3 or shall be determined from the lesser of Eq. 707-5 or Eq. 707-6.

$$B_y = 1070 \sqrt{f'_m A_b} \quad (707-5)$$

$$B_v = 0.12 A_b f_y \quad (707-6)$$

Where the anchor bolt edge distance l_{be} in the direction of load is less than 12 bolt diameters, the value of B_v in Eq. 707-5 shall be reduced by linear interpolation to zero at a l_{be} distance of 40 mm. Where adjacent anchors are spaced closer than $8d_b$, the allowable shear of the adjacent anchors determined by Eq. 707-5 shall be reduced by linear interpolation to 0.75 times the allowable shear value at a center-to-center spacing of four bolt diameters.

Table 707-3 Allowable Shear, B_v , for Embedded Anchor Bolts for Clay and Concrete Masonry, kN^{1,2}

f'_m (MPa)	ANCHOR BOLT DIAMETER (mm)						
	10	12	16	20	22	25	28
10.3	2.14	3.78	5.92	7.92	8.45	9.12	9.7
12.4	2.14	3.78	5.92	8.28	9.35	9.57	10.1
13.8	2.14	3.78	5.92	8.45	9.17	9.79	10.4
17.2	2.14	3.78	5.92	8.45	9.70	10.4	11.0
20.7	2.14	3.78	5.92	8.45	10.1	10.9	11.5
27.6	2.14	3.78	5.92	8.45	10.9	11.7	12.4
34.4	2.14	3.78	5.92	8.45	11.5	12.3	13.1
41.3	2.14	3.78	5.92	8.45	11.6	12.9	13.7

¹ Values are for bolts of at least A 307 quality. Bolts shall be those specified in Section 706.2.14.1.

² Values shown are for work with or without special inspection.

707.1.5.4 Combined Shear and Tension

Anchor bolts subjected to combined shear and tension shall be designed in accordance with:

$$\frac{b_t}{B_t} + \frac{b_v}{B_v} \leq 1.0 \quad (707-7)$$

707.1.6 Compression in Walls and Columns**707.1.6.1 Walls, Axial Loads**

Stresses due to compressive forces applied at the centroid of wall may be computed, assuming uniform distribution over the effective area, by

$$f_a = P/A_e \quad (707-8)$$

707.1.6.2 Columns, Axial Loads

Stresses due to compressive forces applied at the centroid of columns may be computed by Eq. 707-8 assuming uniform distribution over the effective area.

707.1.6.3 Columns, Bending or Combined Bending and Axial Loads

Stresses in columns due to combined bending and axial loads shall satisfy the requirements of Section 707.2.7 where f_a/F_a is replaced by P/P_a . Columns subjected to bending shall meet all applicable requirements for flexural design.

707.1.7 Shear Walls, Design Loads

When calculating shear or diagonal tension stresses, shear walls which resist seismic forces in Seismic Zone 4 shall be designed to resist 1.5 times the forces required by Section 208.5.

707.1.8 Design, Composite Construction**707.1.8.1 General**

The requirements of this section govern multi-wythe masonry in which at least one wythe has strength or composition characteristics different from the other wythe or wythes and is adequately bonded to act as a single structural element.

The following assumptions shall apply to the design of composite masonry:

1. Analysis shall be based on elastic transformed section of the net area.
2. The maximum computed stress in any portion of composite masonry shall not exceed the allowable stress for the material of that portion.

707.1.8.2 Determination of Modulus of Elasticity

The modulus of elasticity of each type of masonry in composite construction shall be measured by tests if the modular ratio of the respective types of masonry exceeds 2 to 1 as determined by Section 706.2.12.

707.1.8.3 Structural Continuity**707.1.8.3.1 Bonding of Wythes**

All wythes of composite masonry elements shall be tied together as specified in Section 706.1.5.2 as a minimum requirement. Additional ties or the combination of grout and metal ties shall be provided to transfer the calculated stress.

707.1.8.3.2 Material Properties

The effect of dimensional changes of the various materials and different boundary conditions of various wythes shall be included in the design.

707.1.8.4 Design Procedure, Transformed Sections

In the design of transformed sections, one material is chosen as the reference material, and the other materials are transformed to an equivalent area of the reference material by multiplying the areas of the other materials by the respective ratios of the modulus of elasticity of the other materials to that of the reference material. Thickness of the transformed area and its distance perpendicular to a given bending axis remain unchanged. Effective height or length of the element remains unchanged.

707.1.9 Reuse of Masonry Units

The allowable working stresses for reused masonry units shall not exceed 50 percent of those permitted for new masonry units of the same properties.

707.2 Design of Reinforced Masonry

707.2.1 Scope

The requirements of this section are in addition to the requirements of Sections 706 and 707.1, and govern masonry in which reinforcement is used to resist forces.

Walls with openings used to resist lateral loads whose pier and beam elements are within the dimensional limits of Section 708.2.6.1.2 may be designed in accordance with Section 708.2.6. Walls used to resist lateral loads not meeting the dimensional limits of Section 708.2.6.1.2 may be designed as walls in accordance with this section or Section 708.2.5.

707.2.2 Reinforcement

707.2.2.1 Maximum Reinforcement Size

The maximum size of reinforcement shall be 32 mm. Maximum reinforcement area in cells shall be 6 percent of the cell area without splices and 12 percent of the cell area with splices.

707.2.2.2 Cover

All reinforcing bars, except joint reinforcement, shall be completely embedded in mortar or grout and have a minimum cover, including the masonry unit, of at least 20 mm, 40 mm of cover when the masonry is exposed to weather and 50 mm of cover when the masonry is exposed to soil.

707.2.2.3 Development Length

The required development length l_d for deformed bars or deformed wire shall be calculated by:

$$l_d = 0.29d_b f_s \quad \text{for bars in tension} \quad (707-9)$$

$$l_d = 0.22d_b f_s \quad \text{for bars in compression} \quad (707-10)$$

Development length for smooth bars shall be twice the length determined by Eq. 707-9.

707.2.2.4 Reinforcement Bond Stress

Bond stress u in reinforcing bars shall not exceed the following:

Plain Bars	410 kPa
Deformed Bars	1370 kPa
Deformed Bars without Special Inspection	690 kPa

707.2.2.5 Hooks

- The term "standard hook" shall mean one of the following:
 - A 180-degree turn plus extension of at least 4 bar diameters, but not less than 63 mm at free end of bar.
 - A 90-degree turn plus extension of at least 12 bar diameters at free end of bar.
 - For stirrup and tie anchorage only, either a 90-degree or a 135-degree turn, plus an extension of at least six bar diameters, but not less than 65 mm at the free end of the bar.
- Inside diameter of bend of the bars, other than for stirrups and ties, shall not be less than that set forth in Table 407-2.
- Inside diameter of bend for 16 mm or smaller stirrups and ties shall not be less than four bar diameter. Inside diameter of bend for 16 mm or larger stirrups and ties shall not be less than that set forth in Table 407-2.
- Hooks shall not be permitted in the tension portion of any beam, except at the ends of simple or cantilever beams or at the freely supported end of continuous or restrained beams.
- Hooks shall not be assumed to carry a load which would produce a tensile stress in the bar greater than 52 MPa.
- Hooks shall not be considered effective in adding to the compressive resistance of bars.
- Any mechanical device capable of developing the strength of the bar without damage to the masonry may be used in lieu of a hook. Data must be presented to show the adequacy of such devices.

707.2.2.6 Splices

The amount of lap of lapped splices shall be sufficient to transfer the allowable stress of the reinforcement as specified in Sections 706.3.4, 707.2.2.3 and 707.2.12. In no case shall the length of the lapped splice be less than 30 bar diameters for compression or 40 bar diameters for tension.

Welded or mechanical connections shall develop 125 percent of the specified yield strength of the bar in tension.

Exception:

For compression bars in columns that are not part of the seismic-resisting system and are not subject to flexure, only the compressive strength need be developed.

When adjacent splices in grouted masonry are separated by 76 mm or less, the required lap length shall be increased 30 percent.

Exception:

Where lap splices are staggered at least 24 bars diameters, no increase in lap length is required.

See Section 707.2.12 for lap splice increases.

707.2.3 Design Assumptions

The following assumptions are in addition to those stated in Section 707.1.4:

1. Masonry carries no tensile stress.
2. Reinforcement is completely surrounded by and bonded to masonry material so that they work together as a homogenous material within the range of allowable working stresses.

707.2.4 Nonrectangular Flexural Elements

Flexural elements of nonrectangular cross section shall be designed in accordance with the assumptions given in Sections 707.1.4 and 707.2.3.

707.2.5 Allowable Axial Compressive Stress and Force

For members other than reinforced masonry columns, the allowable axial compressive stress F_a shall be determined as follows:

for $h'/r \leq 99$

$$F_a = 0.25f'_m \left[1 - \left(\frac{h'}{140r} \right)^2 \right] \quad (707-11)$$

for $h'/r > 99$

$$F_a = 0.25f'_m \left(\frac{70r}{h'} \right)^2 \quad (707-12)$$

For reinforced masonry columns, the allowable axial compressive force P_a shall be determined as follows:

for $h'/r \leq 99$

$$P_a = [0.25f'_m A_e + 0.65A_s F_{sc}] \left[1 - \left(\frac{h'}{140r} \right)^2 \right] \quad (707-13)$$

for $h'/r > 99$

$$P_a = [0.25f'_m A_e + 0.65A_s F_{sc}] \left[\frac{70r}{h'} \right] \quad (707-14)$$

707.2.6 Allowable Flexural Compressive Stress

The allowable flexural compressive stress F_b is:

$$F_b = 0.33f'_m, \quad 13.8 \text{ MPa maximum} \quad (707-15)$$

707.2.7 Combined Compressive Stresses, Unity Equation

Elements subjected to combined axial and flexural stresses shall be designed in accordance with accepted principles of mechanics or in accordance with Eq. 707-16:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \quad (707-16)$$

707.2.8 Allowable Shear Stress in Flexural Members

Where no shear reinforcement is provided, the allowable shear stress F_v in flexural members is:

$$F_v = 0.083\sqrt{f'_m}, \quad 0.345 \text{ MPa maximum} \quad (707-17)$$

Exception:

For a distance of 1/16 the clear span beyond the point of inflection, the maximum stress shall be 140 kPa.

Where shear reinforcement designed to take entire shear force is provided, the allowable shear stress, F_v in flexural members is:

$$F_v = 0.25\sqrt{f'_m}, \quad 1.0 \text{ MPa maximum} \quad (707-18)$$

707.2.9 Allowable Shear Stress in Shear Walls

Where inplane flexural reinforcement is provided and masonry is used to resist all shear, the allowable shear stress F_v in shear wall is:

$$F_v = \frac{1}{36} \left(4 - \frac{M}{Vd} \right) \sqrt{f'_m} \left(80 - 45 \frac{M}{Vd} \right) \text{ maximum} \quad (707-19)$$

$$\text{For } M/Vd \geq 1, F_v = 1/12 \sqrt{f'_m}, \quad 0.24 \text{ MPa maximum} \quad (707-20)$$

Where shear reinforcement designed to take all the shear is provided, the allowable shear stress F_v in shear walls is:

$$\text{For } M/Vd < 1,$$

$$F_v = \frac{1}{24} \left(4 - \frac{M}{Vd} \right) \sqrt{f'_m}, \left(120 - 45 \frac{M}{Vd} \right) \text{ maximum} \quad (707-21)$$

$$M/Vd \geq 1, F_v = 0.12\sqrt{f'_m}, \quad 0.52 \text{ MPa maximum} \quad (707-22)$$

707.2.10 Allowable Bearing Stress

When a member bears on the full area of a masonry element, the allowable bearing stress F_{br} is:

$$F_{br} = 0.26f'_m \quad (707-23)$$

When a member bears on one third or less of a masonry element, the allowable bearing stress F_{br} is:

$$F_{br} = 0.38f'_m \quad (707-24)$$

Eq. 707-24 applies only when the least dimension between the edges of the loaded and unloaded areas is a minimum of one fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one third but less than the full area shall be interpolated between the values of Eqs. 707-23 and 707-24.

707.2.11 Allowable Stresses in Reinforcement

The allowable stresses in reinforcement shall be as follows:

1. Tensile Stress**1.1 Deformed bars,**

$$F_s = 0.5f_y, \quad 165 \text{ MPa maximum} \quad (707-25)$$

1.2 Wire reinforcement,

$$F_s = 0.5f_y, \quad 200 \text{ MPa maximum} \quad (707-26)$$

1.3 Ties, anchors and smooth bars,

$$F_s = 0.4f_y, \quad 140 \text{ MPa maximum} \quad (707-27)$$

2. Compressive Stress**2.1 Deformed bars in columns,**

$$F_{sc} = 0.4f_y, \quad 165 \text{ MPa maximum} \quad (707-28)$$

2.2 Deformed bars in flexural members,

$$F_s = 0.5f_y, \quad 165 \text{ MPa maximum} \quad (707-29)$$

- 2.3 Deformed bars in shear walls which are confined by lateral ties throughout the distance where compression reinforcement is required and where such lateral ties are not less than 6 mm in diameter and spaced not farther apart than 16 bar diameters or 48 tie diameters,

$$F_{sc} = 0.4f_y, \quad 165 \text{ MPa maximum} \quad (707-30)$$

707.2.12 Lap Splice Increases

In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tensile stress F_s , the lap length of splices shall be increased not less than 50 percent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 percent increase may be used.

707.2.13 Reinforcement for Columns

Columns shall be provided with reinforcement as specified in this section.

707.2.13.1 Vertical Reinforcement

The area of vertical reinforcement shall not be less than $0.005A_e$ and not more than $0.04A_e$. At least four 10 mm bars shall be provided. The minimum clear distance between parallel bars in columns shall be two and one half times the bar diameter.

707.2.14 Compression in Walls and Columns

707.2.14.1 General

Stresses due to compressive forces in walls and columns shall be calculated in accordance with Section 707.2.5.

707.2.14.2 Walls, Bending or Combined Bending and Axial Loads

Stresses in walls due to combined bending and axial loads shall satisfy the requirements of Section 707.2.7 where f_a is given by Eq. 707-8. Walls subjected to bending with or without axial loads shall meet all applicable requirements for flexural design. The design of walls with a h'/t ratio larger than 30 shall be based on forces and moments determined from an analysis of the structure. Such analysis shall consider the influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, effect of deflections on moments and forces and the effects of duration of loads.

707.2.15 Flexural Design, Rectangular Flexural Elements

Rectangular elements shall be designed in accordance with the following Equations or other methods based on the assumptions given in Sections 707.1.4, 707.2.3 and this section.

1. Compressive stress in the masonry:

$$f_b = \frac{M}{bd^2} \left(\frac{2}{jk} \right) \quad (707-31)$$

2. Tensile stress in the longitudinal reinforcement:

$$f_s = \frac{M}{A_s j d} \quad (707-32)$$

3. Design coefficients:

$$k = \sqrt{(np)^2 + 2np} - np \quad (707-33)$$

or

$$k = \frac{1}{1 + \frac{f_s}{nf_b}} \quad (707-34)$$

$$j = 1 - \frac{k}{3} \quad (707-35)$$

707.2.16 Bond of Flexural Reinforcement

In flexural members in which tensile reinforcement is parallel to the compressive face, the bond stress shall be computed by the Equation:

$$u = \frac{V}{\Sigma_o j d} \quad (707-36)$$

707.2.17 Shear in Flexural Members and Shear Walls

The shear stress in flexural members and shear walls shall be computed by:

$$f_v = \frac{V}{bjd} \quad (707-37)$$

For members of T or I section, b' shall be substituted for b . Where f_v as computed by Eq. 707-37 exceeds the allowable shear stress in masonry, F_v , web reinforcement shall be provided and designed to carry the total shear force. Both vertical and horizontal shear stresses shall be considered.

$$F_{br} = 0.38f'_m \quad (707-46)$$

Eq. 707-46 applies only when the least dimension between the edges of the loaded and unloaded areas is a minimum of one fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one third but less than the full area shall be interpolated between the values of Equations 707-45 and 707-46.

707.3.9 Combined Bending and Axial Loads, Compressive Stresses

Compressive stresses due to combined bending and axial loads shall satisfy the requirements of Section 707.3.4.

707.3.10 Compression in Walls and Columns

Stresses due to compressive forces in walls and columns shall be calculated in accordance with Section 707.2.5.

707.3.11 Flexural Design

Stresses due to flexure shall not exceed the values given in Sections 707.1.2, 707.3.3 and 707.3.5, where:

$$f_b = Mc/I \quad (707-47)$$

707.3.12 Shear in Flexural Members and Shear Walls

Shear calculations for flexural members and shear walls shall be based on Eq. 707-48.

$$f_v = V/A_e \quad (707-48)$$

707.3.13 Corbels

The slope of corbelling (angle measured from the horizontal to the face of the corbelled surface) or unreinforced masonry shall not be less than 60°.

The maximum horizontal projection of corbelling from the plane of the wall shall be such that allowable stresses are not exceeded.

707.3.14 Stack Bond

Masonry units laid in stack bond shall have longitudinal reinforcement of at least 0.00027 times the vertical cross-sectional area of the wall placed horizontally in the bed joints or in bond beams spaced vertically not more than 1.20 m apart.

SECTION 708 STRENGTH DESIGN OF MASONRY

708.1 General

708.1.1 General Provisions

The design of hollow-unit clay and concrete masonry structures using strength design shall comply with the provisions of Section 706 and this section.

Exception:

Two-wythe solid-unit masonry may be used under Sections 708.2.1 and 708.2.4.

708.1.2 Quality Assurance Provisions

Special inspection during construction shall be provided as set forth in Section 1701.5, Item 7 of UBC.

708.1.3 Required Strength

The required strength shall be determined in accordance with the factored load combinations of Section 203.3.

708.1.4 Design Strength

Design strength is the nominal strength, multiplied by the strength-reduction factor, ϕ , as specified in this section. Masonry members shall be proportioned such that the design strength exceeds the required strength.

708.1.4.1 Beams, Piers and Columns

708.1.4.1.1 Flexure

Flexure with or without axial load, the value of ϕ shall be determined from Eq. 708-1:

$$\phi = 0.8 - \frac{P_u}{A_e f'_m} \quad (708-1)$$

and $0.60 \leq \phi \leq 0.80$

708.1.4.1.2 Shear

Shear: $\phi = 0.60$.

708.1.4.2 Wall Design for Out-of-Plane Load**708.1.4.2.1 Walls with Factored Axial Load of $0.04 f'_m A_g$ or less**

Flexure: $\phi = 0.80$.

708.1.4.2.2 Walls with Factored Axial Load Greater than $0.04 f'_m A_g$

Axial load and axial load with flexure: $\phi = 0.80$.

Shear: $\phi = 0.60$.

708.1.4.3 Wall Design for in-Plane Loads**708.1.4.3.1 Axial Load**

Axial load and axial load with flexure: $\phi = 0.65$.

For walls with symmetrical reinforcement in which f_y does not exceed 420 MPa, the value of ϕ may be increased linearly to 0.85 as the value of ϕP_n decreases from $0.10 f'_m A_e$ or $0.25 P_b$ to zero.

For solid grouted walls, the value of P_b may be calculated by Eq. 708-2

$$P_b = 0.85 f'_m b a_b \quad (708-2)$$

where:

$$a_b = 0.85 d \{ e_{mu} / [e_{mu} + (f_y / E_s)] \} \quad (708-3)$$

708.1.4.3.2 Shear

Shear: $\phi = 0.60$.

The value of ϕ may be 0.80 for any shear wall when its nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength for the factored-load combination.

708.1.4.4 Moment-Resisting Wall Frames**708.1.4.4.1 Flexure With or Without Axial Load**

The value of ϕ shall be as determined from Eq. 708-4; however, the value of ϕ shall not be less than 0.65 nor greater than 0.85.

$$\phi = 0.85 - 2 \left(\frac{P_u}{A_n f'_m} \right) \quad (708-4)$$

708.1.4.4.2 Shear

Shear: $\phi = 0.80$.

708.1.4.5 Anchor

Anchor bolts: $\phi = 0.80$.

708.1.4.6 Reinforcement**708.1.4.6.1 Development**

Development: $\phi = 0.80$.

708.1.4.6.2 Splices

Splices: $\phi = 0.80$.

708.1.5 Anchor Bolts**708.1.5.1 Required Strength**

The required strength of embedded anchor bolts shall be determined from factored loads as specified in Section 708.1.3.

708.1.5.2 Nominal Anchor Bolt Strength

The nominal strength of anchor bolts times the strength-reduction factor shall equal or exceed the required strength.

The nominal tensile capacity of anchor bolts shall be determined from the lesser of Eq. 708-5 or 708-6.

$$B_{tn} = 0.084 A_p \sqrt{f'_m} \quad (708-5)$$

$$B_{tn} = 0.4 A_b f_y \quad (708-6)$$

The area A_p shall be the lesser of Eq. 708-7 or 708-8 and where the projected areas of adjacent anchor bolts overlap, the value of A_p of each anchor bolt shall be reduced by one half of the overlapping area.

$$A_p = \pi l_{b2}^2 \quad (708-7)$$

$$A_p = \pi l_{be2}^2 \quad (708-8)$$

The nominal shear capacity of anchor bolts shall be determined from the lesser of Eq. 708-9 or 708-10.

$$B_{sn} = 2750 \sqrt[4]{f'_m A_b} \quad (708-9)$$

$$B_{sn} = 250 A_b f_y \quad (708-10)$$

Where the anchor bolt edge distance, l_{be} , in the direction of load is less than 12 bolt diameters, the value of B_{tn} in Eq. 808-9 shall be reduced by linear interpolation to zero at an l_{be} distance of 38 mm. Where adjacent anchor bolts are spaced closer than $8d_b$, the nominal shear strength of the adjacent anchors determined by Eq. 708-9 shall be reduced by linear interpolation to 0.75 times the nominal shear strength at a center-to-center spacing of four bolt diameters.

Anchor bolts subjected to combined shear and tension shall be designed in accordance with Eq. 708-11.

$$\frac{b_{tu}}{\phi B_{tn}} + \frac{b_{su}}{\phi B_{sn}} \leq 1.0 \quad (708-11)$$

708.1.5.3 Anchor Bolt Placement

Anchor bolts shall be placed so as to meet the edge distance, embedment depth and spacing requirements of Sections 706.2.14.2, 706.2.14.3 and 706.2.14.4.

708.2 Reinforced Masonry

708.2.1 General

708.2.1.1 Scope

The requirements of this section are in addition to the requirements of Sections 706 and 708.1 and govern masonry in which reinforcement is used to resist forces.

708.2.1.2 Design Assumptions

The following assumptions apply:

Masonry carries no tensile stress greater than the modulus of rupture.

Reinforcement is completely surrounded by and bonded to masonry material so that they work together as a homogeneous material.

Nominal strength of singly reinforced masonry wall cross sections for combined flexure and axial load shall be based on applicable conditions of equilibrium and compatibility of strains. Strain in reinforcement and masonry walls shall be assumed to be directly proportional to the distance from the neutral axis.

Maximum usable strain, ϵ_{mu} , at the extreme masonry compression fiber shall:

1. be 0.003 for the design of beams, piers, columns and walls.

2. not exceed 0.003 for moment-resisting wall frames, unless lateral reinforcement as defined in Section 708.2.6.2.6 is utilized.

Strain in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis.

Stress in reinforcement below specified yield strength f_y for grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y , stress in reinforcement shall be considered independent of strain and equal to f_y .

Tensile strength of masonry walls shall be neglected in flexural calculation of strength, except when computing requirements for deflection.

Relationship between masonry compressive stress and masonry strain may be assumed to be rectangular as defined by the following:

Masonry stress of $0.85 f'_m$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = 0.85c$ from the fiber of maximum compressive strain. Distance c from fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

708.2.2 Reinforcement Requirements and Details

708.2.2.1 Maximum Reinforcement

The maximum size of reinforcement shall be 28 mm. The diameter of a bar shall not exceed one fourth the least dimension of a cell. No more than two bars shall be placed in a cell of a wall or a wall frame.

708.2.2.2 Placement

The placement of reinforcement shall comply with the following:

In columns and piers, the clear distance between vertical reinforcing bars shall not be less than one and one-half times the nominal bar diameter, nor less than 40 mm.

708.2.2.3 Cover

All reinforcing bars shall be completely embedded in mortar or grout and shall have a cover of not less than 38 mm nor less than $2.5 d_b$.

708.2.2.4 Standard Hooks

A standard hook shall conform to Section 407.2.

708.2.2.5 Minimum Bend Diameter for Reinforcing Bars

A standard bend diameter shall conform to Section 407.2.

708.2.2.6 Development

The calculated tension or compression reinforcement shall be developed in accordance with the following provisions:

The embedment length of reinforcement shall be determined by Eq. 708-12.

$$l_d = l_{de}/\phi \quad (708-12)$$

where

$$l_{de} = \frac{1.8d_b^2 f_v}{\sqrt{Kf'_m}} \leq 52d_b \quad (708-13)$$

K shall not exceed $3d_b$.

The minimum embedment length of reinforcement shall be 300 mm.

708.2.2.7 Splices

Reinforcement splices shall comply with one of the following:

1. The minimum length of lap for bars shall be 300 mm or the length determined by Eq. 708-14.

$$l_d = l_{de}/\phi \quad (708-14)$$

Bars spliced by non-contact lap splices shall be spaced transversely a distance not greater than one fifth the required length of lap or more than 200 mm.

2. A welded splice shall have the bars butted and welded to develop in tension 125 percent of the yield strength of the bar, f_y .
3. Mechanical splices shall have the bars connected to develop in tension or compression, as required, at least 125 percent of the yield strength of the bar, f_y .

708.2.3 Design of Beams, Piers and Columns**708.2.3.1 General**

The requirements of this section are for the design of masonry beams, piers and columns.

The value of f'_m shall not be less than 10 MPa. For computational purposes, the value of f'_m shall not exceed 28 MPa.

708.2.3.2 Design Assumptions

Member design forces shall be based on an analysis which considers the relative stiffness of structural members. The calculation of lateral stiffness shall include the contribution of all beams, piers and columns.

The effects of cracking on member stiffness shall be considered.

The drift ratio of piers and columns shall satisfy the limits specified in Chapter 2.

708.2.3.3 Balanced Reinforcement Ratio for Compression Limit State.

Calculation of the balanced reinforcement ratio, ρ_b , shall be based on the following assumptions:

1. The distribution of strain across the section shall be assumed to vary linearly from the maximum usable strain, ϵ_{mu} , at the extreme compression fiber of the element, to a yield strain of f_y/E_s at the extreme tension fiber of the element.
2. Compression forces shall be in equilibrium with the sum of tension forces in the reinforcement and the maximum axial load associated with a loading combination **1.0D + 1.0L + (1.4E or 1.3W)**.
3. The reinforcement shall be assumed to be uniformly distributed over the depth of the element and the balanced reinforcement ratio shall be calculated as the area of this reinforcement divided by the net area of the element.
4. All longitudinal reinforcement shall be included in calculating the balanced reinforcement ratio except that the contribution of compression reinforcement to resistance of compressive loads shall not be considered.

708.2.3.4 Required Strength

Except as required by Sections 708.2.3.6 through 708.2.3.12, the required strength shall be determined in accordance with Section 708.1.3.

708.2.3.5 Design Strength

Design strength provided by beam, pier or column cross sections in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ , specified in Section 708.1.4.

708.2.3.6 Nominal Strength**708.2.3.6.1 Nominal Axial and Flexural Strength**

The nominal axial strength, P_n , and the nominal flexural strength, M_n , of a cross section shall be determined in accordance with the design assumptions of Section 708.2.1.2 and 708.2.3.2.

The maximum nominal axial compressive strength shall be determined in accordance with Eq. 708-15.

$$P_n = 0.80[0.85f'_m(A_e - A_s) + f_y A_s] \quad (708-15)$$

708.2.3.6.2 Nominal Shear Strength

The nominal shear strength shall be:

$$V_n = V_m + V_s \quad (708-16)$$

where

For **SI**

$$V_m = 0.083C_d A_e \sqrt{f'_m}, 63C_d A_e \text{ maximum} \quad (708-17)$$

$$\text{and } V_s = A_e \rho_n f_y \quad (708-18)$$

1. The nominal shear strength shall not exceed the value given in Table 708-1.

Table 708-1 Maximum Nominal Shear Strength Values ^{1,2}

M/V_d	V_n maximum
≤ 0.25	$6.0A_e\sqrt{f'_m} \leq 380A_e \left(322A_e\sqrt{f'_m} \right) \leq 1691A_e$
≥ 1.00	$4.0A_e\sqrt{f'_m} \leq 250A_e \left(214A_e\sqrt{f'_m} \right) \leq 1113A_e$

¹ M is the maximum bending moment that occurs simultaneously with the shear load V at the section under consideration. Interpolation may be by straight line for M/V_d values between 0.25 and 1.00.

² V_n is in N, and f'_m is in kPa.

2. The value of V_m shall be assumed to be zero within any region subjected to net tension factored loads.

3. The value of V_m shall be assumed to be 170 kPa where M_u is greater than $0.7M_n$. The required moment, M_u , for seismic design for comparison with the $0.7M_n$ value of this section shall be based on an R_w of 3.

708.2.3.7 Reinforcement

1. Where transverse reinforcement is required, the maximum spacing shall not exceed one half the depth of the member nor 1200 mm.
2. Flexural reinforcement shall be uniformly distributed throughout the depth of the element.
3. Flexural elements subjected to load reversals shall be symmetrically reinforced.
4. The nominal moment strength at any section along a member shall not be less than one fourth of the maximum moment strength.
5. The flexural reinforcement ratio, ρ , shall not exceed $0.5\rho_b$.
6. Lap splices shall comply with the provisions of Section 708.2.2.7.
7. Welded splices and mechanical splices which develop at least 125 percent of the specified yield strength of a bar may be used for splicing the reinforcement. Not more than two longitudinal bars shall be spliced at a section. The distance between splices of adjacent bars shall be at least 750 mm along the longitudinal axis.
8. Specified yield strength of reinforcement shall not exceed 420 MPa. The actual yield strength based on mill tests shall not exceed 1.25 times the specified yield strength.

708.2.3.8 Seismic Design Provisions

The lateral seismic load resistance in any line or story level shall be provided by shear walls or wall frames, or a combination of shear walls and wall frames. Shear walls and wall frames shall provide at least 80 percent of the lateral stiffness in any line or story level.

Exception:

Where seismic loads are determined based on R_w not greater than three and where all joints satisfy the provisions of Section 708.2.6.2.9, the piers may be used to provide seismic load resistance.

708.2.3.9 Dimensional Limits

Dimensions shall be in accordance with the following:

1. Beams

- 1.1 The nominal width of a beam shall not be less than 150 mm.
- 1.2 The clear distance between locations of lateral bracing of the compression side of the beam shall not exceed 32 times the least width of the compression area.
- 1.3 The nominal depth of a beam shall not be less than 200 mm.

2. Piers

- 2.1 The nominal width of a pier shall not be less than 153 mm and shall not exceed 400 mm.
- 2.2 The distance between lateral supports of a pier shall not exceed 30 times the nominal width of the piers except as provided for in Section 708.2.3.9, Item 2.3.
- 2.3 When the distance between lateral supports of a pier exceeds 30 times the nominal width of the pier, the provisions of Section 708.2.4 shall be used for design.
- 2.4 The nominal length of a pier shall not be less than three times the nominal width of the pier. The nominal length of a pier shall not be greater than six times the nominal width of the pier. The clear height of a pier shall not exceed five times the nominal length of the pier.

Exception:

The length of a pier may be equal to the width of the pier when the axial force at the location of maximum moment is less than $0.04f'_m A_g$.

3. Columns

- 3.1 The nominal width of a column shall not be less than 300 mm.
- 3.2 The distance between lateral supports of a column shall not exceed 30 times the nominal width of the column.
- 3.3 The nominal length of a column shall not be less than 300 mm and not greater than three times the nominal width of the column.

708.2.3.10 Beams**708.2.3.10.1 Scope**

Members designed primarily to resist flexure shall comply with the requirements of this section. The factored axial compressive force on a beam shall not exceed $0.05A_e f'_m$.

708.2.3.10.2 Longitudinal Reinforcement

1. The variation in the longitudinal reinforcing bars shall not be greater than one bar size. Not more than two bar sizes shall be used in a beam.
2. The nominal flexural strength of a beam shall not be less than 1.3 times the nominal cracking moment strength of the beam. The modulus of rupture, f_r , for this calculation shall be assumed to be 1.6 MPa.

708.2.3.10.3 Transverse Reinforcement

Transverse reinforcement shall be provided where V_u exceeds V_m . Required shear, V_u , shall include the effects of the drift. The value of V_u shall be based on Δ_M . When transverse shear reinforcement is required, the following provisions shall apply:

1. Shear reinforcement shall be a single bar with 180-degree hook at each end.
2. Shear reinforcement shall be hooked around the longitudinal reinforcement.
3. The min. transverse shear reinforcement ratio shall be 0.0007.
4. The first transverse bar shall not be more than one fourth of the beam depth from the end of the beam.

708.2.3.10.4 Construction

Beams shall be solid grouted.

708.2.3.11 Piers**708.2.3.11.1 Scope**

Piers proportioned to resist flexure and shear in conjunction with axial load shall comply with the requirements of this section. The factored axial compression on the piers shall not exceed $0.3A_e f'_m$.

708.2.3.11.2 Longitudinal Reinforcement

A pier subjected to in-plane stress reversals shall be longitudinally reinforced symmetrically on both sides of the neutral axis of the pier.

1. One bar shall be provided in the end cells.
2. The minimum longitudinal reinforcement ratio shall be 0.0007.

708.2.3.11.3 Transverse Reinforcement

Transverse reinforcement shall be provided where V_u exceeds V_m . Required shear, V_u , shall include the effects of drift. The value of V_u shall be based on Δ_M . When transverse shear reinforcement is required, the following provisions shall apply:

1. Shear reinforcement shall be hooked around the extreme longitudinal bars with a 180-degree hook. Alternatively, at wall intersections, transverse reinforcement with a 90-degree standard hook around a vertical bar in the intersecting wall shall be permitted.
2. The minimum transverse reinforcement ratio shall be 0.0015.

708.2.3.12 Columns**708.2.3.12.1 Scope**

Columns shall comply with the requirements of this section.

708.2.3.12.2 Longitudinal Reinforcement

Longitudinal reinforcement shall be a minimum of four bars, one in each corner of the column.

1. Maximum reinforcement area shall be $0.03A_e$.
2. Minimum reinforcement area shall be $0.005A_e$.

708.2.3.12.3 Lateral Ties

1. Lateral ties shall be provided in accordance with Section 706.3.6.
2. Minimum lateral reinforcement area shall be $0.0018A_g$.

708.2.3.12.4 Construction

Columns shall be solid grouted.

708.2.4 Wall Design for Out-of-Plane Loads**708.2.4.1 General**

The requirements of this section are for the design of walls for out-of-plane loads.

708.2.4.2 Maximum Reinforcement

The reinforcement ratio shall not exceed $0.5\rho_b$.

708.2.4.3 Moment and Deflection Calculations

All moment and deflection calculations in Section 708.2.4 are based on simple support conditions top and bottom. Other support and fixity conditions, moments and deflections shall be calculated using established principles of mechanics.

708.2.4.4 Walls with Axial Load of $0.04 f'_m A_g$ or less

The procedures set forth in this section, which consider the slenderness of walls by representing effects of axial forces and deflection in calculation of moments, shall be used when the vertical load stress at the location of maximum moment does not exceed $0.04 f'_m$ as computed by Eq. 708-19. The value of f'_m shall not exceed 40 MPa.

$$\frac{P_w + P_f}{A_g} \leq 0.04 f'_m \quad (708-19)$$

Walls shall have a minimum thickness of 150 mm.

Required moment and axial force shall be determined at the mid-height of the wall and shall be used for design. The factored moment, M_u , at the mid-height of the wall shall be determined by Eq. 708-20.

$$M_u = \frac{w_u h^2}{8} + P_{uf} \frac{e}{2} + P_u \Delta_u \quad (708-20)$$

where

Δ_u = deflection at mid-height of wall due to factored loads

$$P_u = P_{uw} + P_{uf} \quad (708-21)$$

The design strength for out-of-plane wall loading shall be determined by Eq. 708-22.

$$M_u \leq \phi M_n \quad (708-22)$$

where

$$M_n = A_{se} f_y (d - a/2) \quad (708-23)$$

$$A_{se} = (A_s f_y + P_u) / f_y, \text{ effective area of steel} \quad (708-24)$$

$$a = (P_u + A_s f_y) / 0.85 f'_m b, \text{ depth of stress block due to factored loads} \quad (708-25)$$

708.2.4.5 Wall with Axial Load Greater than $0.04 f'_m A_g$

The procedures set forth in this section shall be used for the design of masonry walls when the vertical load stresses at the location of maximum moment exceed $0.04 f'_m$ but are less than $0.2 f'_m$ and the slenderness ratio h'/t does not exceed 30.

Design strength provided by the wall cross section in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ , specified in Section 708.1.4. Walls shall be proportioned such that the design strength exceeds the required strength.

The nominal shear strength shall be determined by Eq. 808-26.

$$V_n = 0.166 A_{mv} \sqrt{f'_m} \quad (708-26)$$

708.2.4.6 Deflection Design

The mid-height deflection, Δ_s , under service lateral and vertical loads (without load factors) shall be limited by the relation:

$$\Delta_s = 0.007h \quad (708-27)$$

$P\Delta$ effects shall be included in deflection calculation. The midheight deflection shall be computed with the following Eq.:

$$\Delta_s = \frac{5M_s h^2}{48E_m I_g} \quad \text{for} \quad M_{ser} \leq M_{cr} \quad (708-28)$$

$$\Delta_s = \frac{5M_{cr} h^2}{48E_m I_g} + \frac{5(M_{ser} - M_{cr}) h^2}{48E_m I_{cr}}$$

$$\text{for} \quad M_{cr} < M_{ser} < M_n \quad (708-29)$$

The cracking moment strength of the wall shall be determined from the Eq.:

$$M_{cr} = S f_r \quad (708-30)$$

The modulus of rupture, f_r , shall be as follows:

1. For fully grouted hollow-unit masonry,

$$f_r = 0.33 \sqrt{f'_m}, \quad 1.6 \text{ MPa maximum} \quad (708-31)$$

2. For partially grouted hollow-unit masonry,

$$f_r = 0.21 \sqrt{f'_m}, \quad 0.86 \text{ MPa maximum} \quad (708-32)$$

3. For two-wythe brick masonry,

$$f_r = 0.166 \sqrt{f'_m}, \quad 0.86 \text{ MPa maximum} \quad (708-33)$$

708.2.5 Wall Design for In-Plane Loads

708.2.5.1 General

The requirements of this section are for the design of walls for in-plane loads.

The value of f'_m shall not be less than 10 MPa nor greater than 28 MPa.

708.2.5.2 Reinforcement

Reinforcement shall be in accordance with the following:

1. Minimum reinforcement shall be provided in accordance with Section 706.1.12.4, Item 2.3, for all seismic areas using this method of analysis.
2. When the shear wall failure mode is in flexure, the nominal flexural strength of the shear wall shall be at least 1.8 times the cracking moment strength of a fully grouted wall or 3.0 times the cracking moment strength of a partially grouted wall from Eq. 708-30.
3. The amount of vertical reinforcement shall not be less than one half the horizontal reinforcement.
4. Spacing of horizontal reinforcement within the region defined in Section 708.2.5.5, Item 3, shall not exceed three times the nominal wall thickness nor 600 mm.

708.2.5.3 Design Strength

Design strength provided by the shear wall cross section in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ , specified in Section 708.1.4.3.

708.2.5.4 Axial Strength

The nominal axial strength of the shear wall supporting axial loads only shall be calculated by Eq. 708-34.

$$P_o = 0.85f'_m(A_e - A_s) + f_y A_s \quad (708-34)$$

Axial design strength provided by the shear wall cross section shall satisfy Eq. 708-35.

$$P_u \leq 0.80\phi P_o \quad (708-35)$$

708.2.5.5 Shear Strength

Shear strength shall be as follows:

1. The nominal shear strength shall be determined using either Item 2 or 3 below. Maximum nominal shear strength values are determined from Table 708-1.
2. The nominal shear strength of the shear wall shall be determined from Eq. 708-36, except as provided in Item 3 below

$$V_n = V_m + V_s \quad (708-36)$$

where

$$V_m = 0.083C_d A_{mv} \sqrt{f'_m} \quad (708-37)$$

and

$$V_s = A_{mv} \rho_n f_y \quad (708-38)$$

3. For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.

For all cross sections within the region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall, the nominal shear strength shall be determined from Eq. 708-39.

$$V_n = A_{mv} \rho_n f_y \quad (708-39)$$

The required shear strength for this region shall be calculated at a distance $L_w/2$ above the base of the shear wall, but not to exceed one half storey height.

For the other region, the nominal shear strength of the shear wall shall be determined from Eq. 708-36.

708.2.5.6 Boundary Members

Boundary members shall be as follows:

1. Boundary members shall be provided at the boundaries of shear walls when the compressive strains in the wall exceed 0.0015. The strain shall be determined using factored forces and R_w equal to 1.5.
2. The minimum length of the boundary member shall be three times the thickness of the wall, but shall include all areas where the compressive strain per Section 2108.2.6.2.7 is greater than 0.0015.
3. Lateral reinforcement shall be provided for the boundary elements. The lateral reinforcement shall be a minimum of 10 mm diameter at a maximum of 200 mm spacing within the grouted core or equivalent confinement which can develop an ultimate compressive masonry strain of at least 0.006.

708.2.6 Design of Moment-Resisting Wall Frames**708.2.6.1 General Requirements****708.2.6.1.1 Scope**

The requirements of this section are for the design of fully grouted moment-resisting wall frames constructed of reinforced open-end hollow-unit concrete or hollow-unit clay masonry.

708.2.6.1.2 Dimensional Limits

Dimensions shall be in accordance with the following:

708.2.6.1.2.1 Beams

Clear span for the beam shall not be less than two times its depth.

The nominal depth of the beam shall not be less than two units or 400 mm, whichever is greater. The nominal beam depth to nominal beam width ratio shall not exceed 6.

The nominal width of the beam shall be the greater of 200 mm or 1/26 of the clear span between pier faces.

708.2.6.1.2.2 Piers

The nominal depth of piers shall not exceed 2.4 m. Nominal depth shall not be less than two full units or 800 mm, whichever is greater.

The nominal width of piers shall not be less than the nominal width of the beam, nor less than 200 mm or 1/14 of the clear height between beam faces, whichever is greater.

The clear height-to-depth ratio of pier shall not exceed 5.

708.2.6.1.2.3 Analysis

Member design forces shall be based on an analysis which considers the relative stiffness of pier and beam member, including the stiffening influence of joints.

The calculation of beam moment capacity for the determination of pier design shall include any contribution of floor slab reinforcement.

The out-of-plane drift ratio of all piers shall satisfy the drift ratio limits specified in Section 2-47.

708.2.6.2 Design Procedure**708.2.6.2.1 Required Strength**

Except as required by the Sections 708.2.6.7 and 708.2.6.2.8, the required strength shall be determined in accordance with Section 708.1.3

708.2.6.2.2 Design Strength

Design strength provided by frame member cross sections in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ , specified in Section 708.1.4.4

Members shall be proportioned such that the design strength exceeds the required strength.

708.2.6.2.3 Design Assumption for Nominal Strength

The nominal strength of member cross sections shall be based on assumptions prescribed in Section 708.2.1.2.

The value of f'_m shall not be less than 10 MPa or greater than 28 MPa.

708.2.6.2.4 Reinforcement

The nominal moment strength at any section along a member shall not be less than one fourth of the higher moment strength provided at the two ends of the member.

Lap splices shall be as defined in Section 708.2.2.7. The center of the lap splice shall be the center of the member clear length.

Welded splices and mechanical connection conforming to Section 412.14.3. Item 1 through 4 of UBC, may be used for splicing the reinforcement at any section provided not more than alternate longitudinal bars are spliced at section, and the distance between splices of alternate bars is at least 600 mm along the longitudinal axis.

Reinforcement shall not have a specified yield strength greater than 420 MPa. The actual yield strength based on mill tests shall not exceed the specified yield strength times 1.25.

708.2.6.2.5 Flexural Members (Beam)

Requirements of this section apply to beams proportioned primarily to resist flexure as follows.

The axial compressive force on beams due to factored loads shall not exceed $0.10A_n f'_m$.

1. Longitudinal Reinforcement

At any section of a beam, each masonry unit through the beam depth shall contain longitudinal reinforcement.

The variation in the longitudinal reinforcement area between units at any section shall not be greater than 50 percent, except multiple diam. 12 bars shall not be greater than 100 percent of the minimum area of longitudinal reinforcement contained by any one unit, except where splices occur.

Minimum reinforcement ratio calculated over the gross cross section shall be 0.002.

Maximum reinforcement ratio calculated over the gross cross section shall be $0.15f'_m/f_y$.

2. Transverse Reinforcement

Transverse reinforcement shall be hooked around top and bottom longitudinal bars with a standard 180-degree hook, as defined in Section 708.2.2.4, and shall be single pieces.

Within an end region extending one beam depth from pier faces and at any region at which beam flexural yielding may occur during seismic or wind loading, maximum spacing of transverse reinforcement shall not exceed one fourth the nominal depth of the beam.

The maximum spacing of transverse reinforcement shall not exceed one half the nominal depth of the beam.

Minimum reinforcement ratio shall be 0.0015.

The first transverse bar shall not be more than 100 mm from the face of the pier.

708.2.6.2.6 Members Subjected to Axial Force and Flexure

The requirements set forth in this subsection apply to piers proportioned to resist flexure in conjunction with axial loads.

1. Longitudinal Reinforcement

A minimum of four longitudinal bars shall be provided at all sections of every pier.

Flexural reinforcement shall be distributed across the member depth. Variation in reinforcement area between reinforced cells shall not exceed 50 percent.

Minimum reinforcement ratio calculated over the gross cross section shall be 0.002

Maximum reinforcement ratio calculated over the gross cross section shall be $0.15f'_m/f_y$.

Maximum bar diameter shall be one eighth nominal width of the pier.

2. Transverse Reinforcement

Transverse reinforcement shall be hooked around the extreme longitudinal bars with standard 180-degree hook as defined in Section 708.2.2.4.

Within an end region extending one pier depth from the end of the beam, and at any region at which flexural yielding may occur during seismic or wind loading, the maximum spacing of transverse reinforcement shall not exceed one fourth the nominal depth of the pier.

The maximum spacing of transverse reinforcement shall not exceed one half the nominal depth of the pier.

The minimum transverse reinforcement ratio shall be 0.0015.

3. Lateral Reinforcement

Lateral reinforcement shall be provided to confine the grouted core when compressive strains due to axial and bending forces exceed 0.0015, corresponding to factored forces with R_w equal to 1.5. The unconfined portion of the cross section with strain exceeding 0.0015 shall be neglected in computing the nominal strength of the section.

The total cross-sectional area of rectangular tie reinforcement for the confined core shall not be less than:

$$A_{sh} = 0.09sh_c f'_m / f_{yh} \quad (708-40)$$

Alternatively, equivalent confinement which can develop an ultimate compressive strain of at least 0.006 may be substituted for rectangular tie reinforcement.

708.2.6.2.7 Pier Design Forces

Pier nominal moment strength shall not be less than 1.6 times the pier moment corresponding to the development of the beam plastic hinges, except at the foundation level.

Pier axial load based on the development of beam plastic hinges in accordance with the paragraph above and including factored dead and live loads shall not exceed $0.15A_n f'_m$.

The drift ratio of piers shall satisfy the limits specified in Chapter 2.

The effects of cracking on member stiffness shall be considered.

The base plastic hinge of the pier must form immediately adjacent to the level of lateral support provided at the base or foundation.

708.2.6.2.8 Shear Design**1. General**

Beam and pier nominal shear stress shall not be less than 1.4 times the shears corresponding to the development of the flexural yielding.

It shall be assumed in the calculation of member shear force that moments of opposite sign act at the joint faces and that the member is loaded with the tributary gravity load along its span.

2. Vertical Member Shear Strength

The nominal shear stress shall be determined from Eq. 708-41.

$$V_n = V_m + V_s \quad (708-41)$$

where

$$V_m = 0.083 C_d A_{mv} \sqrt{f'_m} \quad (708-42)$$

and

$$V_s = A_{mv} \rho_n f_y \quad (708-43)$$

The value of V_m shall be zero within an end region extending one pier depth from beam faces and at any region where pier flexural yielding may occur during seismic loading, and at piers subjected to net tension factored loads. The nominal pier shear strength, V_n , shall not exceed the value determined from Table 708-1.

3. Beam Shear Stress

The nominal shear stress shall be determined from Eq. 708-44.

$$V_m = 0.01 A_{mv} \sqrt{f'_m} \quad (708-44)$$

The value of V_m shall be zero within an end region extending one beam depth from pier faces and to any region at which beam flexure yielding may occur during seismic loading.

The nominal beam shear stress, V_n , shall be determined from Eq. 708-45.

$$V_v \leq 0.33 A_{mv} \sqrt{f'_m} \quad (708-45)$$

708.2.6.2.9 Joints**1. General Requirements**

Where reinforcing bars extend through a joint, the joint dimensions shall be proportioned such that

$$h_p > 57827 d_{bb} / f'_g \quad (708-46)$$

and

$$h_p > 21685 d_{bb} / f'_g \quad (708-47)$$

The grout stress shall not exceed 35 MPa for the purposes of Eq. 708-46 and 708-47.

Joint shear forces shall be calculated on the assumption that the stress in all flexural tension reinforcement of the beams at that pier faces is $1.4 f_y$.

Strength of joint shall be governed by the appropriate strength reduction factors specified in Section 708.1.4.4.

Beam longitudinal reinforcement terminating in a pier shall be extended to the far face of the pier and anchored by a standard 90 or 180 degree hook, as defined in Section 708.2.2.4, bent back to the beam.

Pier longitudinal reinforcement terminating in a beam shall be extended to the far face of the beam and anchored by a standard 90 or 180 degree hook, as defined in Section 708.2.2.4, bent back to the beam.

2. Transverse Reinforcement

Special horizontal joint shear reinforcement crossing a potential corner to corner diagonal joint shear crack, and anchored by standard hooks, as defined in Section 708.2.2.4, around the extreme pier reinforcing bars shall be provided such that

$$A_{jh} = 0.5 V_{jh} / f_y \quad (708-48)$$

Vertical shear forces may be considered to be carried by a combination of masonry shear resisting mechanisms and truss mechanism involving intermediate pier reinforcing bars.

3. Shear Stress

The nominal horizontal shear stress of the joint shall not exceed:

$$0.58 \sqrt{f'_m} \text{ or } 2.5 \text{ MPa, whichever is less.}$$

SECTION 709 SEISMIC DESIGN

709.1 Scope

The seismic design requirements of this section apply to the design of masonry and the construction of masonry building elements, except glass unit masonry, when subjected to earthquake loads. NSCP Vol. 1, Chapter 2, has been cited here as the appropriate reference for the distribution of seismic forces in order to avoid confusion in the event that the general building code has no provisions or is inconsistent with the type of distribution upon which these provisions are based.

709.2 General

Masonry structures and masonry elements in the seismic zones shown in Chapter 2, Figure 208-1 shall be designed in accordance with the design requirements of this chapter and the special provisions for each seismic zone given in Sections 709.3 through 709.7. In addition, masonry structures and masonry elements shall comply with either the requirements of Section 706 or the requirements of Section 709.2.1.

709.2.1 Strength Requirement

For masonry structures that are not designed in accordance with Section 706, the provisions of this section shall apply. The design strength of masonry structures and masonry elements shall be at least equal to the required strength determined in accordance with this section, except for masonry structures and masonry elements in areas lower than zone 2 designed in accordance with the provisions of Section 710.

709.2.1.1 Required Strength

Required strength, U , to resist the seismic forces in such combinations with gravity and other loads, including load factors, shall be as required in the earthquake loads section of NSCP Vol. 1 Section 208, except that nonbearing masonry walls shall be designed for the seismic force applied perpendicular to the plane of the wall and uniformly distributed over the wall area in lieu of the provisions of ASCE 7 Section 9.8.1.1.

709.2.1.2 Nominal Strength

The nominal strength of masonry shall be taken as 2.5 times the allowable stress value. The allowable stress values shall be determined in accordance with Section 707.2 or Section 707.3 and are permitted to be increased by one-third $\left(\frac{1}{3}\right)$ for load combinations including earthquake.

709.2.1.3 Design Strength

The design strength of masonry provided by a member, its connections to other members and its cross sections in terms of flexure, axial load, and shear shall be taken as the nominal strength multiplied by a strength reduction factor, ϕ .

- | | |
|--|---------------|
| (a) Axial load and flexure except for flexural tension in unreinforced masonry | $\phi = 0.80$ |
| (b) Flexural tension in unreinforced masonry | $\phi = 0.40$ |
| (c) Shear | $\phi = 0.60$ |
| (d) Shear and tension in anchor bolts embedded in masonry | $\phi = 0.60$ |

709.2.1.4 Drift Limits

The calculated storey drift of masonry structures due to the combination of seismic forces and gravity loads shall not exceed 0.007 times the storey height.

709.3 Special Provisions for Seismic Zone 2

Masonry structures in seismic zone 2 shall comply with the requirements of Sections 707, 708 and 710.

709.3.1 Anchorage of Masonry Walls

Masonry walls shall be anchored to the roof and all floors that provide lateral support for the wall. The anchorage shall provide a direct connection between the walls and the floor or roof construction. The connections shall be capable of resisting the greater of a seismic lateral force induced by the wall or 14590 times the effective peak velocity-related acceleration, N/m of wall.

709.4 Additional Requirements for Seismic Zone 2

Structures in Zone 2 shall comply with the requirements of Section 709.3 and to the additional requirements of this section.

709.4.1 Design of Elements that are Not Part of Lateral Force-Resisting System

709.4.1.1 Load-Bearing Frames

Load bearing frames or columns that are not part of the lateral force resisting system shall be analyzed as to their effect on the response of the system. Such frames or

columns shall be adequate for vertical load carrying capacity and induced moment due to the design storey drift.

709.4.1.2 Masonry Walls and Elements

Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design storey drift.

709.4.1.3 Reinforcement Requirements for Masonry Elements

Masonry elements listed in Section 709.4.1.2 shall be reinforced in either the horizontal or vertical direction in accordance with the following:

709.4.1.3.1 Horizontal Reinforcement

Horizontal joint reinforcement shall consist of at least two longitudinal MW11 wires spaced not more than 400 mm for walls greater than 100 mm in width and at least one longitudinal MW11 wire spaced not more than 400 mm for walls not exceeding 100 mm in width; or at least one 12 mm diameter bar spaced not more than 1.2 m. Where two longitudinal wires of joint reinforcement are used, the space between these wires shall be the widest that the mortar joint will accommodate. Horizontal reinforcement shall be provided within 400 mm of the top and bottom of these masonry elements.

709.4.1.3.2 Vertical Reinforcement

Vertical reinforcement shall consist of at least one 12 mm diameter bar spaced not more than 1.2 m. Vertical reinforcement shall be located within 400 mm of the ends of masonry walls.

709.4.2 Design of Elements that are Part of the Lateral Force - Resisting System

709.4.2.1 Connections to Masonry Shear Walls

Connectors shall be provided to transfer forces between masonry walls and horizontal elements in accordance with the requirements of Section 706. Connectors shall be designed to transfer horizontal design forces acting either perpendicular or parallel to the wall, but not less than 2.9 kN/m of wall. The maximum spacing between connectors shall be 1.2 m.

709.4.2.2 Connections to Masonry Columns

Connectors shall be provided to transfer forces between masonry columns and horizontal elements in accordance with the requirements of Section 706. Where anchor bolts are used to connect horizontal elements to the tops of columns, anchor bolts shall be placed within lateral ties. Lateral ties shall enclose both the vertical bars in the column and the anchor bolts. There shall be a minimum of two 12 mm diameter lateral ties provided in the top 125 mm of the column.

709.4.2.3 Minimum Reinforcement Requirements for Masonry Shear Walls

Vertical reinforcement of at least 129 mm^2 in cross-sectional area shall be provided at comers, within 400 mm of each side of openings, within 200 mm of each side of movement joints, within 200 mm of the ends of walls, and at a maximum spacing of 3.0 m.

Horizontal joint reinforcement shall consist of at least two MW11 wires spaced not more than 400 mm; or bond beam reinforcement shall be provided of at least 129 mm^2 in cross-sectional area spaced not more than 3.0 m. Horizontal reinforcement shall also be provided at the bottom and top of wall openings and shall extend not less than 600 mm nor less than 40 bar diameters past the opening; continuously at structurally connected roof and floor levels; and within 400 mm of the top of walls.

709.5 Special Provisions for Seismic Zone 4

Masonry structures in seismic zone 4 shall comply with the requirements of Section 709.3 and to the additional requirements of this section.

709.5.1 Design Requirements

Masonry elements other than those covered by Section 709.4.1.2 shall be designed in accordance with the requirements of Sections 707.2 and 708.2.

709.5.2 Minimum Reinforcement Requirements for Masonry Walls

Masonry walls other than those covered by Section 709.4.1.3 shall be reinforced in both the vertical and horizontal direction. The sum of the cross-sectional area of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall, and the minimum cross-sectional area in each direction shall be not less than 0.0007 times the gross cross-sectional area of the wall. Reinforcement shall be uniformly distributed. The maximum spacing of reinforcement shall be 1.2 m provided that the walls are solid grouted and constructed of hollow open-end units, hollow units laid with full head joints or two wythes of solid units. The maximum spacing of reinforcement shall be 600 mm for all other masonry.

709.5.2.1 Shear Wall Reinforcement Requirements

The maximum spacing of vertical and horizontal reinforcement shall be the smaller of; one-third the length of the shear wall, one-third the height of the shear wall, 1.2 m. The minimum cross-sectional area of vertical reinforcement shall be one-third of the required shear reinforcement. Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook.

709.5.3 Minimum Reinforcement for Masonry Columns

Lateral ties in masonry columns shall be spaced not more than 200 mm on center and shall be at least 10 mm diameter. Lateral ties shall be embedded in grout.

709.5.4 Material Requirements

Neither Type N mortar nor masonry cement shall be used as part of the lateral force resisting system.

709.5.5 Lateral Tie Anchorage

Standard hooks for lateral tie anchorage shall be either a 135 degree standard hook or a 180 degree standard hook.

709.6 Additional Requirements for Seismic Zone 4

All masonry structures in seismic zone 4 shall comply with the requirements of Section 709.5 and to the additional requirements of this section.

709.6.1 Design of Elements that are Not Part of Lateral Force Resisting System

Stack bond masonry that is not part of the lateral force-resisting system shall have a horizontal cross sectional area of reinforcement of at least 0.0015 times the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 600 mm. These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

709.6.2 Design of Elements that are Part of Lateral Force Resisting System

Stack bond masonry that is part of the lateral force-resisting system shall have a horizontal cross sectional area of reinforcement of at least 0.0025 times the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 400 mm. These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

SECTION 710 EMPIRICAL DESIGN OF MASONRY

710.1 Height

Building relying on masonry walls for lateral load resistance shall not exceed 10 m in height.

710.2 Lateral Stability

Where the structure depends on masonry walls for lateral stability, shear walls shall be provided parallel to the direction of the lateral forces resisted.

Minimum nominal thickness on masonry shear walls shall be 200 mm.

In each direction in which shear walls are required for lateral stability, the minimum cumulative length of shear walls provided shall be 0.4 times the dimension of the building. The cumulative length of shear walls shall not include openings.

The maximum spacing of shear walls shall not exceed the ratio listed in Table 710-1.

Table 710-1 Shear Wall Spacing Requirements for Empirical Design of Masonry

FLOOR OR ROOF CONSTRUCTION	MAXIMUM RATIO
	Shear Wall Spacing to Shear Wall Length
Cast-in-place concrete	5:1
Precast Concrete	4:1
Metal deck with concrete fill	3:1
Metal deck with no fill	2:1
Wood Diaphragm	2:1

710.3 Compressive Stresses

710.3.1 General

Compressive stresses in masonry due to vertical dead loads plus live loads, excluding wind or seismic loads, shall be determined in accordance with Section 710.4.3. Dead and live loads shall be in accordance with this code with permitted live load reductions.

710.3.2 Allowable Stresses

The compressive stresses in masonry shall not exceed the values set forth in Table 710-2. The allowable stresses given in Table 710-2 for the weakest combination of the units and mortar used in any load wythe shall be used for all loaded wythes of multi-wythe walls.

710.3.3 Stress Calculations

Stresses shall be calculated based on specified rather than nominal dimensions. Calculated compressive stresses shall be determined by dividing the design load by the gross cross-sectional area of the member. The area of openings, chases or recesses in walls shall not be included in the gross cross-sectional area of the wall.

710.3.4 Anchor Bolts

Bolt values shall not exceed those set forth in Table 710-3.

710.4 Lateral Support

Masonry walls shall be laterally supported in either the horizontal or vertical direction not exceeding the intervals set forth in Table 710-4.

Lateral support shall be provided by cross walls, pilasters, buttresses or structural framing members horizontally or by floors, roof or structural framing members vertically.

Except for parapet walls, the ratio of height to nominal thickness for cantilever walls shall not exceed 6 for solid masonry or 4 for hollow masonry.

In computing the ratio for cavity walls, the value of thickness shall be the sums of the nominal thickness of the inner and outer wythes of the masonry. In walls composed of different classes of units and mortars, the ratio of height or length to thickness shall not exceed that allowed for the weakest of the combinations of units and mortar of which the member is composed.

Table 710-2 Allowable Compressive Stresses for Empirical Design of Masonry

CONSTRUCTION: COMPRESSIVE STRENGTH OF UNIT, GROSS AREA	ALLOWABLE COMPRESSIVE STRESSES GROSS CROSS-SECTIONAL AREA (MPa)	
	Type M or S Mortar	Type N Mortar
Solid masonry of brick and other solid units of clay or shale; Sand-lime or concrete brick:		
55.1 plus, MPa	2.41	2.07
31.0 MPa	1.55	1.38
17.2 MPa	1.10	0.96
10.3 MPa	0.79	0.69
Grouted masonry, of clay or shale; sand-lime or concrete:		
31.0 plus, MPa	1.89	1.38
17.2 MPa	1.48	0.96
10.3 MPa	1.21	0.69
Solid masonry of solid concrete masonry units:		
20.7 plus, MPa	1.55	1.38
13.8 MPa	1.10	0.96
8.27 MPa	0.79	0.69
Masonry of hollow load-bearing units:		
13.8 plus, MPa	0.96	0.83
10.3 MPa	0.79	0.69
6.89 MPa	0.52	0.48
4.82 MPa	0.41	0.38
Hollow walls (cavity or masonry bonded) ² solid units:		
17.2 plus, MPa	1.10	0.96
10.3 MPa	0.79	0.69
Hollow units	0.52	0.48
Stone ashlar masonry:		
Granite	4.96	4.41
Limestone or marble	3.10	2.76
Sandstone or cast stone	2.48	2.20
Rubble stone masonry		
Coarse, rough or random	0.83	0.69
Unburned clay masonry	0.21	—

¹ Linear interpolation may be used for determining allowable stresses for masonry units having compressive strengths which are intermediate between those given in the table.

² Where floor and floor loads are carried upon wythe, the gross cross-sectional area is that of the wythe under load. If both wythes are loaded, the gross cross-sectional area is that of the wall minus the area of the cavity between the wythes.

Table 710-3 Allowable Shear on Bolts for Empirically Designed Masonry Except Unburned Clay Units

DIAMETER BOLT (mm)	EMBEDMENT (mm)	SOLID MASONRY (shear in kN)	GROUTED MASONRY (shear in kN)
12	100	1.56	2.47
16	100	2.22	3.34
20	125	3.34	4.89
22	150	4.45	6.67
25	175	5.56	18.2 ²
28	200	6.67	10.0 ²

¹ An additional 50 mm of embedment shall be provided for anchor bolts located in the top of columns for buildings located in Seismic Zones 2 and 4.

² Permitted only with not less than 17.2 MPa units.

Table 710-4 Wall Lateral Support Requirements for Empirical Design of Masonry

CONSTRUCTION	MAXIMUM l/t or h/t
Bearing walls	
Solid or solid grouted	20
All other	18
Nonbearing walls	
Exterior	18
Interior	36

710.5 Minimum Thickness

710.5.1 General

The nominal thickness of masonry bearing walls in buildings more than one story in height shall not be less than 200 mm. Solid masonry walls in one-story buildings may be of 150 mm nominal thickness when not over 2.7 m in height, provided that when gable construction is used, an additional 1.8 m is permitted to the peak of the gable.

Exception:

The thickness of unreinforced grouted brick masonry walls may be 50mm less than required by this section, but in no case less than 150 mm.

710.5.2 Variation in Thickness

Where a change in thickness due to minimum thickness occurs between floor levels, the greater thickness shall be carried up to the higher floor level.

710.5.3 Decrease in Thickness

Where walls of masonry of hollow units or masonry-bonded hollow walls are decrease in thickness, a course or courses of solid masonry shall be constructed between the walls below and the thinner wall above, or special units or construction shall be used to transmit the loads from face shells or wythes to the walls below.

710.5.4 Parapets

Parapet walls shall be at least 200 mm in thickness and their height shall not exceed three times their thickness. The parapet wall shall not be thinner than the wall below.

710.5.5 Foundation Walls

Mortar used in masonry foundation walls shall be either Type M or S.

Where the height of unbalanced fill (height of finished grade above basement floor or inside grade) and the height of the wall between lateral support does not exceed 2.4 m, and when the equivalent fluid weight of unbalanced fill does not exceed 480 kg/m², the minimum thickness of foundation walls shall be as set forth in Table 710-5. Maximum depths of unbalanced fill permitted in Table 710-5 may be increased with the approval of the building official when local soil conditions warrant such an increase.

Where the height of unbalanced fill, height between lateral supports or equivalent fluid weight of unbalanced fill exceeds that set forth above, foundation walls shall be designed in accordance with Chapter 3.

Table 710-5 Thickness of Foundation Walls for Empirical Design of Masonry

FOUNDATION WALL CONSTRUCTION	NOMINAL THICKNESS (mm)	MAXIMUM DEPTH OF UNBALANCED FILL (m)
Masonry of hollow units, ungrouted	200	1.20
	250	1.50
	300	1.85
Masonry of solid units	200	1.50
	250	1.85
	300	2.15
Masonry of hollow or solid units, fully grouted	200	2.53
	20	2.45
	300	2.45
Masonry of hollow units reinforced vertically with 12 mm bars and grout at 600 mm o.c. Bars located not less than 115 mm from pressure Side of wall	200	2.15

710.6 Bond

710.6.1 General

The facing and backing of multi-wythe masonry walls shall be bonded in accordance with this section.

710.6.2 Masonry Headers

Where the facing and backing of solid masonry construction are bonded by masonry headers, not less than 4 percent of the wall surface of each face shall be composed of headers extending not less than 75 mm into the backing. The distance between adjacent full-length headers shall not exceed 600 mm either vertically or horizontally. In walls in which a single header does not extend through the wall, headers from opposite sides shall overlap at least 75 mm, or headers from opposite sides shall be covered with another header course overlapping the header below at least 75 mm.

Where two or more hollow units are used to make up the thickness of the wall, the stretcher courses shall be bonded at vertical intervals not exceeding 865 mm by lapping at least 75 mm over the unit below, or by lapping at vertical intervals not exceeding 430 mm with units which are at least 50 percent greater in thickness than the units below.

710.6.3 Wall Ties

Where the facing and backing of masonry walls are bonded with 4.8 mm diameter wall ties or metal ties of equivalent stiffness embedded in the horizontal mortar joints, there shall be at least one metal tie for each 0.42 m² of wall area. Ties in alternate courses shall be staggered, the maximum vertical distance between ties shall not exceed 600 mm, and the maximum horizontal distance shall not exceed 900 mm. Rods bent to rectangular shape shall be used with hollow-masonry units laid with the cells vertical. In other walls, the ends of ties shall be bent to 90-degree angles to provide hooks not less than 50 mm long. Additional ties shall be provided at all openings, spaced not more than 900 mm apart around the perimeter and within 300 mm of the opening.

The facing and backing of masonry walls may be bonded with prefabricated joint reinforcement. There shall be at least one cross wire serving as a tie for each 0.25 m² of wall area. The vertical spacing of the joint reinforcement shall not exceed 406 mm. Cross wires of prefabricated joint reinforcement shall be at least 10 mm diameter. The longitudinal reinforcement shall be embedded in mortar.

710.6.4 Longitudinal Bond

In each wythe of masonry, head joints in successive courses shall be offset at least one fourth of the unit length or the walls shall be reinforced longitudinally as required in Section 706.1.12.3, Item 4.

710.7 Anchorage

710.7.1 Intersecting Walls

Masonry walls depending on one another for lateral support shall be anchored or bonded at locations where they meet or intersect by one of the following methods:

1. Fifty percent of the units at the intersection shall be laid in an overlapping pattern, with alternating units having a bearing of not less than 75 mm on the unit below.
2. Walls shall be anchored by steel connectors having a minimum section of 6 mm by 38 mm with ends bent up at least 50 mm, or with cross pins to form anchorage. Such anchors shall be at least 600 mm long and the maximum spacing shall be 1.2 m vertically.
3. Walls shall be anchored by joint reinforcement spaced at a maximum distance of 200 mm vertically. Longitudinal rods of such reinforcement shall be at least 10 mm diameter and shall extend at least 750 mm in each direction at the intersection.

4. Interior nonbearing walls may be anchored at their intersection, at vertical spacing of not more than 400 mm with joint reinforcement or 6 mm mesh galvanized hardware cloth.
5. Other metal ties, joint reinforcement or anchors may be used, provided they are spaced to provide equivalent area of anchorage to that required by this section.

710.7.2 Floor and Roof Anchorage

Floor and roof diaphragms providing lateral support to masonry walls shall be connected to the masonry walls by one of the following methods:

1. Wood floor joists bearing on masonry walls shall be anchored to the wall by approved metal strap anchors at intervals not exceeding 1.8 m. Joists parallel to the wall shall be anchored with metal straps spaced not more than 1.8 m on center extending over and under and secured to at least three joists. Blocking shall be provided between joists at each strap anchor.
2. Steel floor joists shall be anchored to masonry walls with 10 mm diameter bars, or their equivalent, spaced not more than 1.8 m on center. Where joists are parallel to the wall, anchors shall be located at joists cross bridging.
3. Roof structures shall be anchored to masonry walls with 12 mm bolts at 1.8 m on center or their equivalent. Bolts shall extend and be embedded at least 400 mm into the masonry, or be hooked or welded to not less than 130 mm^2 of bond beam reinforcement placed not less than 150 mm from the top of the wall.

710.7.3 Walls Adjoining Structural Framing

Where walls are dependent on the structural frame for lateral support, they shall be anchored to the structural members with metal anchors or keyed to the structural members. Metal anchors shall consist of 12 mm bolts spaced at a maximum of 1.2 m on center and embedded at least 100 mm into the masonry, or their equivalent area.

710.8 Unburned Clay Masonry

710.8.1 General

Masonry of stabilized clay unburned units shall not be used in any building more than one storey in height. The unsupported height of every wall of unburned clay units shall not be more than 10 times the thickness of such walls. Bearing walls shall in no case be less than 400 mm in thickness. All footing walls which support masonry of unburned clay units shall extend to an elevation not less than 150 mm above the adjacent ground at all points.

710.8.2 Bolts

Bolt values shall not exceed those set forth in Table 710-6.

Table 710-6 Allowable Shear on Bolts for Masonry of Unburned Clay Units

DIAMETER OF BOLTS (mm)	EMBEDMENTS (mm)	SHEAR (kN)
16	300	0.89
20	380	1.33
22	457	1.78
25	533	2.22
28	600	2.67

710.9 Stone Masonry

710.9.1 General

Stone masonry is that form of construction made with natural or cast stone in which the units are laid and set in mortar with all joints filled.

710.9.2 Construction

In ashlar masonry, bond stones uniformly distributed shall be provided to the extent of not less than 10 percent of the area of exposed facets. Rubble stone masonry 600 mm or less in thickness shall have bond stones with a maximum spacing of 900 mm vertically and 900 mm horizontally and, if the masonry is of greater thickness than 600 mm, shall have one bond stone for each 0.56 m^2 of wall surface on both sides.

710.9.3 Minimum Thickness

The thickness of stone masonry bearing walls shall not be less than 400 mm.

SECTION 711 GLASS MASONRY

711.1 General

Masonry of glass blocks may be used in non-load-bearing exterior or interior walls and in openings which might otherwise be filled with windows, either isolated or in continuous bands, provided the glass block panels have a minimum thickness of 75 mm at the mortar joint and the mortared surfaces of the blocks are treated for mortar bonding. Glass block may be solid or hollow and may contain inserts.

711.2 Mortar Joints

Glass block shall be laid in Type S or N mortar. Both vertical and horizontal mortar joints shall be at least 6 mm and not more than 10 mm thick and shall be completely filled. All mortar contact surfaces shall be treated to ensure adhesion between mortar and glass.

711.3 Lateral Support

Glass panels shall be laterally supported along each end of the panel.

Lateral support shall be provided by panel anchors spaced not more than 400 mm on center or by channels. The lateral support shall be capable of resisting the horizontal design forces determined in Chapter 2 or a minimum of 3 kN/m of wall, whichever is greater. The connection shall accommodate movement requirements of Section 711.6.

711.4 Reinforcement

Glass block panels shall have joint reinforcement spaced not more than 400 mm on center and located in the mortar bed joint extending the entire length of the panel. A lapping of longitudinal wires for a minimum of 150 mm is required for joint reinforcement splices. Joint reinforcement shall also be placed in the bed joint immediately below and above openings in the panel. Joint reinforcement shall conform to ASTM A 385 and A 641. Joint reinforcement in exterior panels shall be hot-dip galvanized in accordance with ASTM A 385 and A 641.

711.5 Size of Panels

Glass block panels for exterior walls shall not exceed 13.5 m² of unsupported wall surface or 4.50 m in any dimension. For interior walls, glass block panels shall not exceed 23.2 m² of unsupported area or 7.60 m in any dimension.

711.6 Expansion Joints

Glass block shall be provided with expansion joints along the sides and top, and these joints shall have sufficient thickness to accommodate displacements of the supporting structure, but not less than 10 mm. Expansion joints shall be entirely free of mortar and shall be filled with resilient material.

711.7 Reuse of Units

Glass block units shall not be reused after being removed from an existing panel.

Table 711-1 Radius of Gyration^a for Concrete Masonry Units^b, mm (Length of 400 mm)

GROUT SPACING (mm)	NOMINAL WIDTH OF WALL (mm)				
	100	150	200	250	300
Solid Grouted	26	41	56	70	85
400	30	45	62	77	93
600	31	48	64	80	97
800	32	49	66	83	99
1000	32	49	67	84	101
1200	32	50	678	85	102
1400	33	50	68	85	103
1600	33	50	69	86	104
1800	33	51	69	86	104
No grout	34	53	72	90	109

^a For single-wythe masonry or for an individual wythe of a cavity wall.

$$r = \sqrt{I/A_e}$$

^b The radius of gyration shall be based on the specified dimensions of the masonry units or shall be in accordance with the values shown which are based on the minimum dimensions of hollow concrete masonry unit face shells and webs in accordance with UBC Standard 21-4 for two cell units.

Table 711-2 Radius of Gyration¹ for Clay Masonry Unit, mm (Length, 400 mm^b)

GROUT SPACING (mm)	NOMINAL WIDTH OF WALL (mm)				
	100	150	200	250	300
Solid Grouted	27	42	57	71	86
400	29	45	61	77	93
600	30	47	64	80	96
800	31	48	65	81	98
1000	32	49	66	82	99
1200	32	49	66	83	100
1400	32	49	67	83	100
1600	32	50	67	84	101
1800	32	50	67	84	101
No grout	32	51	70	87	105

^a For single-wythe masonry or for an individual wythe of a cavity wall.

$$r = \sqrt{I/A_e}$$

^b The radius of gyration shall be based on the specified dimensions of the masonry units or shall be in accordance with the values shown which are based on the minimum dimensions of hollow clay concrete masonry face shells and webs in accordance with UBC Standard 21-1 for two cell units.

Table 711-3 Radius of Gyration¹ for Clay Masonry Unit Length, 300 mm^b

GROUT SPACING (mm)	NOMINAL WIDTH OF WALL (mm)				
	100	150	200	250	300
Solid Grouted	27	42	57	72	87
300	29	45	61	76	92
450	30	46	63	78	94
600	31	47	64	79	95
750	31	48	64	80	97
900	32	48	65	81	97
1050	32	48	65	81	97
1200	32	48	65	81	98
1350	32	48	65	82	98
1500	32	49	66	82	98
1650	32	49	66	82	99
1800	32	49	66	82	99
No grout	33	50	67	83	100

^a For single-wythe masonry or for an individual wythe of a cavity wall.

$$r = \sqrt{I/A_e}$$

^b The radius of gyration shall be based on the specified dimensions of the masonry units or shall be in accordance with the values shown which are based on the minimum dimensions of hollow clay concrete masonry face shells and webs in accordance with UBC Standard 21-1 for two cell units.

SECTION 712

MASONRY FIREPLACES

712.1 Definition

A masonry fireplace is a fireplace constructed of concrete or masonry. Masonry fireplaces shall be constructed in accordance with this section.

712.2 Footings and Foundations

Footings for masonry fireplaces and their chimneys shall be constructed of concrete or solid masonry at least 300 mm thick and shall extend at least 150 mm beyond the face of the fireplace or foundation wall on all sides. Footings shall be founded on natural undisturbed earth or engineered fill below frost depth. In areas not subjected to freezing, footings shall be at least 300 mm below finished grade.

712.2.1 Ash Dump Cleanout

Cleanout openings, located within foundation walls below fireboxes, when provided, shall be equipped with ferrous metal or masonry doors and frames constructed to remain tightly closed, except when in use. Cleanouts shall be accessible and located so that ash removal will not create a hazard to combustible materials.

712.3 Seismic Reinforcing

Masonry or concrete fireplaces shall be constructed, anchored, supported and reinforced as required in this chapter. In Zone 4, masonry and concrete fireplaces shall be reinforced and anchored as detailed in Sections 712.3.1, 712.3.2, 712.4 and 712.4.1 for chimneys serving fireplaces. In Zone 2, reinforcement and seismic anchorage is not required. In Zone 4, masonry and concrete chimneys shall be reinforced in accordance with the requirements of Sections 701 through 709.

712.3.1 Vertical Reinforcing

For fireplaces with chimneys up to 1.0 m wide, four 10 mm diameter continuous vertical bars, anchored in the foundation, shall be placed in the concrete between wythes of solid masonry or within the cells of hollow unit masonry and grouted in accordance with Section 703.4. For fireplaces with chimneys greater than 1.0 m wide, two additional 12 mm diameter vertical bars shall be provided for each additional 1.0 m in width or fraction thereof.

712.3.2 Horizontal Reinforcing

Vertical reinforcement shall be placed enclosed within 6 mm ties or other reinforcing of equivalent net cross-

sectional area, spaced not to exceed 450 mm on center in concrete; or placed in the bed joints of unit masonry at a minimum of every 450 mm of vertical height. Two such ties shall be provided at each bend in the vertical bars.

712.4 Seismic Anchorage

Masonry and concrete chimneys in Seismic Design Category D shall be anchored at each floor, ceiling or roof line more than 1.8 m above grade, except where constructed completely within the exterior walls. Anchorage shall conform to the following requirements.

712.4.1 Anchorage

4.8 mm by 25 mm straps shall be embedded a minimum of 300 mm into the chimney. Straps shall be hooked around the outer bars and extend 150 mm beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two 12 mm bolts.

712.5 Firebox Walls

Masonry fireboxes shall be constructed of solid masonry units, hollow masonry units grouted solid, stone or concrete. When a lining of firebrick at least 50 mm in thickness or other approved lining is provided, the minimum thickness of back and sidewalls shall each be 200 mm of solid masonry, including the lining. The width of joints between firebricks shall not be greater than 6 mm. When no lining is provided, the total minimum thickness of back and sidewalls shall be 250 mm of solid masonry. Firebrick shall conform to ASTM C 27 or ASTM C 1261 and shall be laid with medium-duty refractory mortar conforming to ASTM C 199.

712.5.1 Steel Fireplace Units

Steel fireplace units are permitted to be installed with solid masonry to form a masonry fireplace provided they are installed according to either the requirements of their listing or the requirements of this section. Steel fireplace units incorporating a steel firebox lining shall be constructed with steel not less than 6 mm in thickness, and an air-circulating chamber which is ducted to the interior of the building. The firebox lining shall be encased with solid masonry to provide a total thickness at the back and sides of not less than 200 mm, of which not less than 100 mm shall be of solid masonry or concrete. Circulating air ducts employed with steel fireplace units shall be constructed of metal or masonry.

712.6 Firebox Dimensions

The firebox of a concrete or masonry fireplace shall have a minimum depth of 500 mm. The throat shall not be less than 200 mm above the fireplace opening. The throat

opening shall not be less than 100 mm in depth. The cross-sectional area of the passageway above the firebox, including the throat, damper and smoke chamber, shall not be less than the cross-sectional area of the flue.

712.7 Lintel and Throat

Masonry over a fireplace opening shall be supported by a lintel of noncombustible material. The minimum required bearing length on each end of the fireplace opening shall be 100 mm. The fireplace throat or damper shall be located a minimum of 200 mm above the top of the fireplace opening.

712.7.1 Damper

Masonry fireplaces shall be equipped with a ferrous metal damper located at least 200 mm above the top of the fireplace opening. Dampers shall be installed in the fireplace or at the top of the flue venting the fireplace, and shall be operable from the room containing the fireplace. Damper controls shall be permitted to be located in the fireplace.

712.8 Smoke Chamber Walls

Smoke chamber walls shall be constructed of solid masonry units, hollow masonry units grouted solid, stone or concrete. Corbeling of masonry units shall not leave unit cores exposed to the inside of the smoke chamber. The inside surface of corbeled masonry shall be parged smooth. Where no lining is provided, the total minimum thickness of front, back and sidewalls shall be 200 mm of solid masonry. When a lining of firebrick at least 50 mm thick, or a lining of vitrified clay at least 16 mm thick, is provided, the total minimum thickness of front, back and sidewalls shall be 150 mm of solid masonry, including the lining. Firebrick shall conform to ASTM C 27 or ASTM C 1261 and shall be laid with refractory mortar conforming to ASTM C 199.

712.8.1 Smoke Chamber Dimensions

The inside height of the smoke chamber from the fireplace throat to the beginning of the flue shall not be greater than the inside width of the fireplace opening. The inside surface of the smoke chamber shall not be inclined more than 45 degrees (0.76 rad) from vertical when prefabricated smoke chamber linings are used or when the smoke chamber walls are rolled or sloped rather than corbeled. When the inside surface of the smoke chamber is formed by corbeled masonry, the walls shall not be corbeled more than 30 degrees (0.52 rad) from vertical.

712.9 Hearth and Hearth Extension

Masonry fireplace hearths and hearth extensions shall be constructed of concrete or masonry, supported by noncombustible materials, and reinforced to carry their own weight and all imposed loads. No combustible material shall remain against the underside of hearths or hearth extensions after construction.

712.9.1 Hearth Thickness

The minimum thickness of fireplace hearths shall be 100 mm.

712.9.2 Hearth Extension Thickness

The minimum thickness of hearth extensions shall be 50 mm.

Exception:

When the bottom of the firebox opening is raised at least 0.20m above the top of the hearth extension, a hearth extension of not less than 10 mm thick brick, concrete, stone, tile or other approved noncombustible material is permitted.

712.10 Hearth Extension Dimensions

Hearth extensions shall extend at least 400 mm in front of, and at least 200 mm beyond, each side of the fireplace opening. Where the fireplace opening is 0.60 m² or larger, the hearth extension shall extend at least 500 mm in front of, and at least 300 mm beyond, each side of the fireplace opening.

712.11 Fireplace Clearance

Any portion of a masonry fireplace located in the interior of a building or within the exterior wall of a building shall have a clearance to combustibles of not less than 50 mm from the front faces and sides of masonry fireplaces and not less than 0.10 m from the back faces of masonry fireplaces. The airspace shall not be filled, except to provide fireblocking in accordance with Section 712.12.

Exceptions:

1. *Masonry fireplaces listed and labeled for use in contact with combustibles in accordance with UL 127 and installed in accordance with the manufacturer's installation instructions are permitted to have combustible material in contact with their exterior surfaces.*
2. *When masonry fireplaces are constructed as part of masonry or concrete walls, combustible materials shall not be in contact with the masonry or concrete walls less than 300 mm from the inside surface of the nearest firebox lining.*
3. *Exposed combustible trim and the edges of sheathing materials, such as wood siding, flooring and drywall, are permitted to abut the masonry fireplace sidewalls and hearth extension, in accordance with Figure 712.11, provided such combustible trim or sheathing is a minimum of 300 mm from the inside surface of the nearest firebox lining.*
4. *Exposed combustible mantels or trim is permitted to be placed directly on the masonry fireplace front surrounding the fireplace opening provided such combustible materials shall not be placed within 150 mm of a fireplace opening. Combustible material directly above and within 300 mm of the fireplace opening shall not project more than 3.2 mm for each 25 mm distance from such opening. Combustible materials located along the sides of the fireplace opening that project more than 40 mm from the face of the fireplace shall have an additional clearance equal to the projection.*

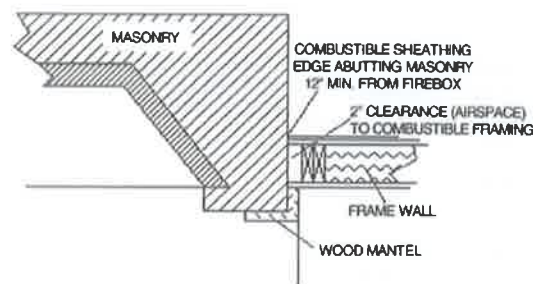


Figure 712.11 Illustration of Exception to Fireplace Clearance Provision

712.12 Fireplace Fireblocking

All spaces between fireplaces and floors and ceilings through which fireplaces pass shall be fireblocked with noncombustible material securely fastened in place. The fireblocking of spaces between wood joists, beams or headers shall be to a depth of 25 mm and shall only be placed on strips of metal or metal lath laid across the spaces between combustible material and the chimney.

712.13 Exterior Air

Factory-built or masonry fireplaces covered in this section shall be equipped with an exterior air supply to ensure proper fuel combustion unless the room is mechanically ventilated and controlled so that the indoor pressure is neutral or positive.

712.13.1 Factory-Built Fireplaces

Exterior combustion air ducts for factory-built fireplaces shall be listed components of the fireplace, and installed according to the fireplace manufacturer's instructions.

712.13.2 Masonry Fireplaces

Listed combustion air ducts for masonry fireplaces shall be installed according to the terms of their listing and manufacturer's instructions.

712.13.3 Exterior Air Intake

The exterior air intake shall be capable of providing all combustion air from the exterior of the dwelling. The exterior air intake shall not be located within the garage, attic, basement or crawl space of the dwelling nor shall the air intake be located at an elevation higher than the firebox. The exterior air intake shall be covered with a corrosion-resistant screen of 6.4 mm mesh.

712.13.4 Clearance

Unlisted combustion air ducts shall be installed with a minimum 25 mm clearance to combustibles for all parts of the duct within 1.5 m of the duct outlet.

712.13.5 Passageway

The combustion air passageway shall be a minimum of 0.040 m^2 and not more than 0.035 m^2 , except that combustion air systems for listed fireplaces or for fireplaces tested for emissions shall be constructed according to the fireplace manufacturer's instructions.

713.13.6 Outlet

The exterior air outlet is permitted to be located in the back or sides of the firebox chamber or within 600 mm of the firebox opening on or near the floor. The outlet shall be closable and designed to prevent burning material from dropping into concealed combustible spaces.

SECTION 713

MASONRY CHIMNEYS

713.1 Definition

A masonry chimney is a chimney constructed of concrete or masonry, hereinafter referred to as "masonry." Masonry chimneys shall be constructed, anchored, supported and reinforced as required in this chapter.

713.2 Footings and Foundations

Footings for masonry chimneys shall be constructed of concrete or solid masonry at least 300 mm thick and shall extend at least 150 mm beyond the face of the foundation or support wall on all sides. Footings shall be founded on natural undisturbed earth or engineered fill below frost depth. In areas not subjected to freezing, footings shall be at least 300 mm below finished grade.

713.3 Seismic Reinforcing

Masonry or concrete chimneys shall be constructed, anchored, supported and reinforced as required in this chapter. In Zone 4, masonry and concrete chimneys shall be reinforced and anchored as detailed in Sections 713.3.1, 713.3.2 and 713.4. In Zone 1, reinforcement and seismic anchorage is not required. In Zone 4, masonry and concrete chimneys shall be reinforced in accordance with the requirements of Sections 701 through 709.

713.3.1 Vertical Reinforcing

For chimneys up to 1.0 m wide, four 12 mm diameter continuous vertical bars anchored in the foundation shall be placed in the concrete between wythes of solid masonry or within the cells of hollow unit masonry and grouted in accordance with Section 703.4. Grout shall be prevented from bonding with the flue liner so that the flue liner is free to move with thermal expansion. For chimneys greater than 1.0 m wide, two additional 12 mm vertical bars shall be provided for each additional 1.0 m width or fraction thereof.

713.3.2 Horizontal Reinforcing

Vertical reinforcement shall be placed enclosed within 6 mm ties, or other reinforcing of equivalent net cross-sectional area, spaced not to exceed 450 mm o.c. in concrete, or placed in the bed joints of unit masonry, at a minimum of every 450 mm of vertical height. Two such ties shall be provided at each bend in the vertical bars.

713.4 Seismic Anchorage

Masonry and concrete chimneys and foundations in Zone 4 shall be anchored at each floor, ceiling or roof line more than 1.8 m above grade, except where constructed completely within the exterior walls. Anchorage shall conform to the following requirements.

713.4.1 Anchorage

Two 4.8 mm by 25 mm straps shall be embedded a minimum of 300 mm into the chimney. Straps shall be hooked around the outer bars and extend 150 mm beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two 12 mm bolts.

713.5 Corbeling

Masonry chimneys shall not be corbelled more than half of the chimney's wall thickness from a wall or foundation, nor shall a chimney be corbeled from a wall or foundation that is less than 300 mm in thickness unless it projects equally on each side of the wall, except that on the second story of a two-story dwelling, corbeling of chimneys on the exterior of the enclosing walls is permitted to equal the wall thickness. The projection of a single course shall not exceed one-half the unit height or one-third of the unit bed depth, whichever is less.

713.6 Changes in Dimension

The chimney wall or chimney flue lining shall not change in size or shape within 150 mm above or below where the chimney passes through floor components, ceiling components or roof components.

713.7 Offsets

Where a masonry chimney is constructed with a fireclay flue liner surrounded by one wythe of masonry, the maximum offset shall be such that the centerline of the flue above the offset does not extend beyond the center of the chimney wall below the offset. Where the chimney offset is supported by masonry below the offset in an approved manner, the maximum offset limitations shall not apply. Each individual corbeled masonry course of the offset shall not exceed the projection limitations specified in Section 713.5.

713.8 Additional Load

Chimneys shall not support loads other than their own weight unless they are designed and constructed to support the additional load. Masonry chimneys are permitted to be constructed as part of the masonry walls or concrete walls of the building.

713.9 Termination

Chimneys shall extend at least 600 mm higher than any portion of the building within 3.0 m, but shall not be less than 900 mm above the highest point where the chimney passes through the roof.

713.9.1 Spark Arrestors

Where a spark arrestor is installed on a masonry chimney, the spark arrestor shall meet all of the following requirements:

1. The net free area of the arrestor shall not be less than four times the net free area of the outlet of the chimney flue it serves.
2. The arrestor screen shall have heat and corrosion resistance equivalent to 20 mm diameter galvanized steel or 25 mm diameter stainless steel.
3. Openings shall not permit the passage of spheres having a diameter greater than 12 mm nor block the passage of spheres having a diameter less than 10 mm.
4. The spark arrestor shall be accessible for cleaning and the screen or chimney cap shall be removable to allow for cleaning of the chimney flue.

713.10 Wall Thickness

Masonry chimney walls shall be constructed of concrete, solid masonry units or hollow masonry units grouted solid with not less than 100 mm nominal thickness.

713.10.1 Masonry Veneer Chimneys

Where masonry is used as veneer for a framed chimney, through flashing and weep holes shall be provided as required by Chapter 14 of IBC.

713.11 Flue Lining (Material)

Masonry chimneys shall be lined. The lining material shall be appropriate for the type of appliance connected, according to the terms of the appliance listing and the manufacturer's instructions.

713.11.1 Residential-Type Appliances (General)

Flue lining systems shall comply with one of the following:

1. Clay flue lining complying with the requirements of ASTM C315, or equivalent.
2. Listed chimney lining systems complying with UL 1777.
3. Factory-built chimneys or chimney units listed for installation within masonry chimneys.
4. Other approved materials that will resist corrosion, erosion, softening or cracking from flue gases and condensate at temperatures up to 1,800°F (982°C).

713.11.1.1 Flue Linings for Specific Appliances

Flue linings other than those covered in Section 713.11.1 intended for use with specific appliances shall comply with Sections 713.11.1.2 through 713.11.1.4 and Sections 713.11.2 and 713.11.3.

713.11.1.2 Gas Appliances

Flue lining systems for gas appliances shall be in accordance with the *International Fuel Gas Code*.

713.11.1.3 Pellet Fuel-Burning Appliances

Flue lining and vent systems for use in masonry chimneys with pellet fuel-burning appliances shall be limited to flue lining systems complying with Section 713.11.1 and pellet vents listed for installation within masonry chimneys (see Section 713.11.1.5 for marking).

713.11.1.4 Oil-Fired Appliances Approved for Use with L-Vent

Flue lining and vent systems for use in masonry chimneys with oil-fired appliances approved for use with Type L vent shall be limited to flue lining systems complying with Section 713.11.1 and listed chimney liners complying with UL 641 (see Section 713.11.1.5 for marking).

713.11.1.5 Notice of Usage

When a flue is relined with a material not complying with Section 713.11.1, the chimney shall be plainly and permanently identified by a label attached to a wall, ceiling or other conspicuous location adjacent to where the connector enters the chimney. The label shall include the following message or equivalent language: "This chimney is for use only with (type or category of appliance) that

burns (type of fuel). Do not connect other types of appliances."

713.11.2 Concrete and Masonry Chimneys for Medium-Heat Appliances

713.11.2.1 General

Concrete and masonry chimneys for medium-heat appliances shall comply with Sections 713.1 through 713.5.

713.11.2.2 Construction

Chimneys for medium-heat appliances shall be constructed of solid masonry units or of concrete with walls a minimum of 200 mm thick, or with stone masonry a minimum of 300 mm thick.

713.11.2.3 Lining

Concrete and masonry chimneys shall be lined with an approved medium-duty refractory brick a minimum of 115 mm thick laid on the 115 mm bed in an approved medium-duty refractory mortar. The lining shall start 600 mm or more below the lowest chimney connector entrance. Chimneys terminating 7.5 m or less above a chimney connector entrance shall be lined to the top.

713.11.2.4 Multiple Passageways

Concrete and masonry chimneys containing more than one passageway shall have the liners separated by a minimum 100 mm thick concrete or solid masonry wall.

713.11.2.5 Termination Height

Concrete and masonry chimneys for medium-heat appliances shall extend a minimum of 3.0 m higher than any portion of any building within 7.5 m.

713.11.2.6 Clearance

A minimum clearance of 100 mm shall be provided between the exterior surfaces of a concrete or masonry chimney for medium-heat appliances and combustible material.

713.11.3 Concrete and Masonry Chimneys for High-Heat Appliances

713.11.3.1 General

Concrete and masonry chimneys for high-heat appliances shall comply with Sections 713.1 through 713.5.

713.16.2 Determination of Minimum Area

The minimum net cross-sectional area of the flue shall be determined in accordance with Figure 713.16. A flue size providing at least the equivalent net cross-sectional area shall be used. Cross-sectional areas of clay flue linings are as provided in Tables 713.16(1) and 713.16(2) or as provided by the manufacturer or as measured in the field. The height of the chimney shall be measured from the firebox floor to the top of the chimney flue.

Table 713.16(1) Net Cross-Sectional Area of Round Flue Sizes

Flue Size, Inside Diameter (mm)	Cross-Sectional Area ($\text{mm}^2 \times 10^3$)
150	18
175	24
200	32
250	50
275	58
300	73
380	114
460	164

Table 713.16(1) Net Cross-Sectional Area of Square and Rectangular Flue Sizes

Flue Size, Outside Nominal Dimensions (mm)	Cross-Sectional Area ($\text{mm}^2 \times 10^3$)
114 x 216	15
114 x 330	22
203 x 203	27
216 x 216	32
216 x 305	43
216 x 330	49
305 x 305	66
216 x 457	65
330 x 330	82
305 x 406	84
330 x 457	112
406 x 406	117
406 x 508	143
457 x 457	150
508 x 508	192
508 x 610	216
610 x 610	278

713.17 Inlet

Inlets to masonry chimneys shall enter from the side. Inlets shall have a thimble of fireclay, rigid refractory material or metal that will prevent the connector from pulling out of the inlet or from extending beyond the wall of the liner.

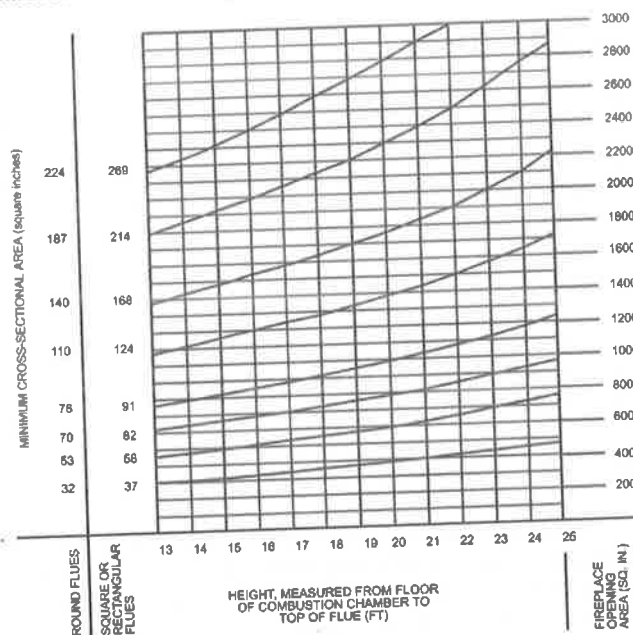


Figure 713.16 Flue Sizes for Masonry Chimneys

713.18 Masonry Chimney Cleanout Openings

Cleanout openings shall be provided within 150 mm of the base of each flue within every masonry chimney. The upper edge of the cleanout shall be located at least 150 mm below the lowest chimney inlet opening. The height of the opening shall be at least 150 mm. The cleanout shall be provided with a noncombustible cover.

Exception:

Chimney flues serving masonry fireplaces, where cleaning is possible through the fireplace opening.

713.19 Chimney Clearances

Any portion of a masonry chimney located in the interior of the building or within the exterior wall of the building shall have a minimum airspace clearance to combustibles of 50 mm. Chimneys located entirely outside the exterior walls of the building, including chimneys that pass through the soffit or cornice, shall have a minimum airspace clearance of 25 mm. The airspace shall not be filled, except to provide fireblocking in accordance with Section 713.20.

Exceptions:

1. *Masonry chimneys equipped with a chimney lining system listed and labeled for use in chimneys in contact with combustibles in accordance with UL 1777, and installed in accordance with the manufacturer's instructions, are permitted to have combustible material in contact with their exterior surfaces.*
2. *Where masonry chimneys are constructed as part of masonry or concrete walls, combustible materials shall not be in contact with the masonry or concrete wall less than 300 mm from the inside surface of the nearest flue lining.*
3. *Exposed combustible trim and the edges of sheathing materials, such as wood siding, are permitted to abut the masonry chimney sidewalls, in accordance with Figure 713.19, provided such combustible trim or sheathing is a minimum of 300 mm from the inside surface of the nearest flue lining. Combustible material and trim shall not overlap the corners of the chimney by more than 25 mm*

713.20 Chimney Fireblocking

All spaces between chimneys and floors and ceilings through which chimneys pass shall be fireblocked with noncombustible material securely fastened in place. The fireblocking of spaces between wood joists, beams or headers shall be to a depth of 25 mm and shall only be placed on strips of metal or metal lath laid across the spaces between combustible material and the chimney.

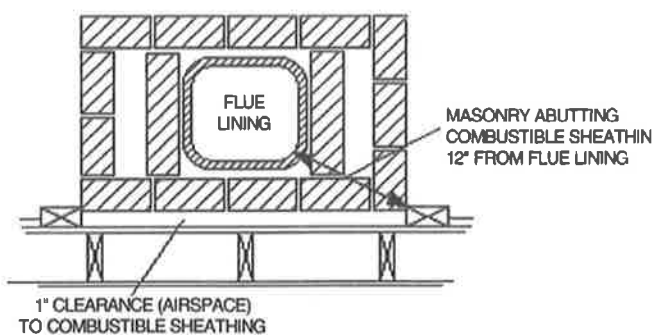


Figure 713.19 Illustration of Exception 3-Chimney Clearance Provision



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